STRUCTURE GEOTECHNICAL REPORT
WHEELER AVENUE OVER INTERSTATE 80 BRIDGE
EX SN 099-0175, PR SN 099-8324
WILL COUNTY, ILLINOIS

For
TranSystems Corporation
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Submitted by
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11. Abstract

A new, two-span bridge will replace the existing four-span bridge carrying Wheeler Avenue over Interstate 80 in Will County, Illinois. The proposed structure will have a back-to-back of abutments length of 244.7 feet and an out-to-out width of 56.0 feet. The proposed north and south abutment cap base elevations are 629.82 and 628.56 feet, respectively, whereas the proposed pier cap base elevation is 607.84 feet. The portion of Wheeler Avenue extending about 130.0 and 90.0 feet, north and south of the bridge, respectively, will be reconstructed. This report provides geotechnical recommendations for the design and construction of the proposed approach embankments, approach slabs, and bridge foundations.

The pavement structure along Wheeler Avenue consists of 3 to 5 inches of asphalt overlying 7 to 12 inches of concrete followed by 2 to 7 inches of aggregate base. Beneath the pavement or at the surface, the general lithologic profile includes up to 5.5 feet of existing embankment fill consisting of stiff to hard silty clay to silty clay loam and clay followed by up to 33.5 feet of stiff to hard silty clay and silty clay loam with discontinuous saturated silt lenses overlying medium dense silty loam and medium dense to very dense sand and sandy gravel. Dolostone bedrock was encountered at elevations of about 593.0 to 584.0 feet. The groundwater level was measured at elevations ranging from 607.0 to 601.0 feet.

The approach embankments behind the north and south abutments will undergo an estimated 0.2 inch of total long-term settlement. Global stability analyses at the embankments show factors of safety meeting the IDOT minimum requirement of 1.5. The maximum factored bearing resistance for the approach slab footings is 2,500 psf.

The bridge abutments and pier could be supported on driven piles. To support the integral abutments, driven 14-inch MSP, 16-inch MSP, HP12x74, and HP14x89 steel piles will provide 100 to 388 kips of factored resistance at total lengths of 26 to 45 feet. At the pier, HP14x89 and HP14x102 steel piles will provide 388 to 446 of factored resistance at total lengths of 18 to 19 feet. We do not anticipate the need for downdrag allowances on the piles. The piers could also be supported on drilled shafts installed either at the top of the bedrock or socketed into the bedrock. Shafts with diameters of 3.0 to 5.0 feet established at the top of bedrock will achieve factored resistances of 700 to 1,950 kips. Rock-socketed shafts would have factored resistances of about 1,040 to 2,900 kips for 3.0- to 5.0-foot diameter sockets.

12. Path to archived file

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TABLE OF CONTENTS

LIST OF TABLES

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STRUCTURE GEOTECHNICAL REPORT
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WILL COUNTY, ILLINOIS
FOR
TRANSYSTEMS CORPORATION

1.0 INTRODUCTION

This report presents the results of our subsurface investigation, laboratory testing, geotechnical evaluations, and recommendations in support of the design and reconstruction of the existing bridge (EX SN 099-0175) carrying Wheeler Avenue over Interstate 80 (I-80) in Will County, Illinois. On the USGS *Joliet Quadrangle 7.5 Minute Series* map, the project is located in the SE 1/4 of Section 17, Tier 35 N, Range 10 E of the Third Principal Meridian (Exhibit 1). Wang Engineering, Inc. (Wang) understands the proposed work will also include the reconstruction of about 130 and 90 feet of the approach roadway north and south of the bridge replacement, respectively. The bridge replacement and roadway reconstruction are part of the proposed widening and reconstruction of I-80 from Houbolt Road to west of Center Street and Larkin Avenue Interchange in Will County, Illinois. The Wheeler Avenue Bridge and roadway will be reconstructed as part of Contract CR-3.

The purpose of this investigation was to characterize the site soil and groundwater conditions, perform geotechnical analyses, and provide recommendations for the design and construction of the proposed bridge foundations and approach slabs. Recommendations pertaining to the Wheeler Avenue roadway reconstruction will be included in the Roadway Geotechnical Report that will be prepared for the I-80 mainline (Contract ML-3).

1.1 Existing Structure and Ground Conditions

Based on the *Bridge Condition Report (BCR)*, dated June 2011 and provided by TranSystems, we understand the existing bridge (SN 099-0175) was originally built in 1966 as a four-span structure with continuous steel stringers that support the deck. The original substructure consists of cast-in-place reinforced concrete stub abutments and multi-column grade separation piers supported on spread footings. The existing bridge has a length of 237.5 feet from back-to-back of abutments and an out-to-out width of 62.0 feet, which accommodates one traffic lane in each direction. Reinforced concrete wingwalls and slope walls are located at the ends of the structure. The structure was repaired in 2002 and the abutments were converted to semi-integral type abutments. The surface elevation at the bridge site is about 640.0 feet along Wheeler Avenue and about 615.0 feet along I-80.

In the project area (see Exhibit 2), about 20 to 40-foot thick overburden made up of moderate plasticity, moderate to high strength, moderate moisture clayey diamicton with occasional silt, sand, and gravel lenses rest over low to moderate plasticity, high strength, and low to moderate moisture content silty clayey diamicton (Bauer et al. 1991, Hansel and Johnson 1996, Willman et al. 1971). The bedrock is made up of dolostone. Top of bedrock is mapped at about 580.0 to 590.0 feet elevation. Sinkholes and other dissolution features are not unknown in the project area (Bretz 1940; Otto 1963). The site is located on the northern, downthrown block of the inactive Sandwich Fault Zone that may be traced five and a half miles southwest of the proposed improvements (Kolata 2005). Records of mining activity in the vicinity of the bridge are missing. Neither the overburden nor the upper bedrock is known to include significant sources of water supply (Woller and Sanderson 1983).

1.2 Proposed Structure

Based on the *General Plan and Elevation (GPE)* provided by AECOM and dated September 16, 2021, Wang understands the existing four-span bridge will be removed and replaced with a new two-span bridge with integral abutments and a pier. The design drawings indicate that the north abutment and pier will be constructed in the same location as the existing ones whereas the south abutment will be constructed about 10.0 feet behind the existing abutment. The new bridge will have a back-to-back of abutments length of 244.7 feet with span lengths of 118.8 and 125.8 feet. The bridge will have an out-to-out width of 56.0 feet to accommodate two 11-foot wide lanes, a 12-foot turn lane, two 5.0-foot wide shoulders, two 5.0-foot wide sidewalks, and two 1.0-foot wide parapets.

Based on the *Plan and Profile* drawing, provided by AECOM and dated August 10, 2021, we understand the profile grade along Wheeler Avenue will be raised by up to 2.0 feet. The *GPE drawing* also shows concrete end slopes graded at 1:2 (V: H). We assume the side slopes would be graded at a slope of 1:2 (V: H), similar to the existing approach side slopes. The *GPE* along with the *Plan and Profile* drawings are included as Appendix E.

2.0 METHODS OF INVESTIGATION

The following sections outline the subsurface and laboratory investigations performed by Wang.

2.1 Field Investigation

The subsurface investigation consisted of three bridge borings, designated as WA-BSB-01 to WA-BSB-03, and two subgrade borings, designated as WA-SGB-01 and WA-SGB-02, drilled by Wang between March 22, 2021 and April 30, 2021. The borings were drilled from elevations of 613.3 to 637.5 feet and were advanced to depths of 11.0 to 67.0 feet bgs. The as-drilled northings and eastings were acquired with a mapping-grade GPS unit. Stations, offsets, and elevations were provided by TranSystems. Boring location data are presented in the *Boring Logs* (Appendix A) and the as-drilled boring locations are shown in the *Boring Location Plan* (Exhibit 3).

A combination of truck- and ATV-mounted drilling rigs equipped with hollow stem augers was used to advance and maintain open boreholes. Mud rotary drilling techniques were used from 10.0 feet bgs to advance the borehole at Boring WA-BSB-03. Soil sampling was performed according to AASHTO T206, *"Penetration Test and Split Barrel Sampling of Soils."* The soil in the bridge borings was sampled at 2.5-foot intervals to 30.0 feet bgs and at 5.0-foot intervals thereafter to the top of sound bedrock. Bedrock cores were obtained from Borings WA-BSB-01 to WA-BSB-03 in 5 to 10-foot runs with an NWD4-sized core barrel. In the subgrade borings, the soil was sampled continuously to the boring termination depths. Soil samples collected from each sampling interval were placed in sealed jars, and rock cores were placed into boxes and transported to the laboratory for further examination and testing.

Field boring logs, prepared and maintained by a Wang field engineer, included lithological descriptions, visual-manual soil (IDH Textural) classifications, results of Rimac and pocket penetrometer unconfined compressive strength tests, and results of Standard Penetration Tests (SPT) recorded as blows per 6 inches of penetration.

Groundwater levels were measured while drilling and at completion of each boring with the exception of Boring WA-BSB-03. Since mud rotary drilling techniques were used to advance and maintain an open borehole at Boring WA-BSB-03, groundwater levels were not available at completion of the boring. Each borehole location was backfilled upon completion with lean grout, soil cuttings, and/or bentonite chips and, where necessary, the pavement surface was restored as much as possible to its original condition.

2.2 Laboratory Testing

The soil samples were tested in the laboratory for moisture content (AASHTO T265). Atterberg limits (AASHTO T89 and T90) and particle size (AASHTO T88) analyses were performed on selected samples. Unconfined compressive strength tests were performed on selected bedrock cores. Field visual descriptions of the soil samples were verified in the laboratory and index tested soils were classified according to the IDH Soil Classification System. The laboratory test results are shown in the *Boring Logs* (Appendix A) and in the *Laboratory Test Results* (Appendix B).

3.0 INVESTIGATION RESULTS

Detailed descriptions of the soil conditions encountered during the subsurface investigation are presented in the attached *Boring Logs* (Appendix A) and in the *Soil Profile* (Exhibit 4). Please note that strata contact lines represent approximate boundaries between soil types. The actual transition between soil types in the field may be gradual in horizontal and vertical directions.

Our subsurface investigation results fit into the local geologic context. The borings drilled in the project area revealed the native sediments consists of silty clay to silty clay loam diamicton (Unit 2) with occasional lenses of silt, sand and gravel (Units 3 & 4) over dolostone bedrock. The top of dolostone bedrock was reached in the structure borings at an elevation of about 584 to 595 feet (20.0 to 52.0 feet bgs) as predicted based on geological data.

3.1 Lithological Profile

Borings WA-BSB-02, WA-SGB-01, and WA-SGB-02 were drilled from Wheeler Avenue and encountered 3 to 5 inches of asphalt pavement overlying 7 to 12 inches of concrete followed by 2 to 7 inches of sandy gravel aggregate base. Boring WA-BSB-01 was drilled just outside the Wheeler Avenue pavement and sampled 4 inches of black silty clay loam topsoil at the surface. Boring WA-BSB-03 was drilled within the grassy median along I-80 and encountered 4 inches of black silty clay loam topsoil. In descending order, the general lithologic succession encountered beneath the pavement or topsoil includes: 1) man-made ground (fill); 2) stiff to hard silty clay to silty clay loam; 3) medium dense silt to silty loam; 4) medium dense to very dense sand to sandy gravel; and 5) medium strong to strong, very poor to poor quality dolostone.

1) Man-made ground (fill)

Beneath topsoil or pavement structure, the borings drilled from Wheeler Avenue encountered up to 5.5 feet of stiff to hard, black, brown, and gray silty clay to silty clay loam and clay fill with unconfined compressive strength (Q_u) values of 1.5 to 4.5 tsf and moisture content values of 8 to

28%. Laboratory index testing showed liquid limit (LL) values of 35 to 46% and plastic limit (PL) values of 15 to 16%.

2) Stiff to hard silty clay to silty clay loam

Beneath the fill, at elevations of 612 to 633 feet, the borings advanced through up to 33.5 feet of stiff to hard, brown to gray silty clay to silty clay loam with discontinuous saturated silt lenses. The silty clay to silty clay loam is characterized by Q_u values of 1.5 to 8.7 tsf and moisture content values of 12 to 24%. Laboratory index testing showed LL values of 33 to 36% and PL values of 15 to 16%. A 12- to 20-inch thick intercalation of loose, saturated silt to silty loam with sand lenses was sampled at elevations of 607 to 604 feet.

3) Medium dense silt to silty loam

At elevations of 601 to 599 feet, borings sampled up to 2.5 feet of medium dense, gray, saturated silt to silty loam with N-values of 19 to 20 blows per foot and moisture content values of 21 to 23%.

4) Medium dense to very dense sand to sandy gravel

At elevations of 599 to 598 feet, the borings advanced through 3.0 to 10.0 feet of medium dense to very dense, brown and damp medium to coarse sand and sandy gravel. It has N-values of 20 blows per foot to refusal and moisture content values of 4 to 7%.

At elevations of 593 to 589 feet, Borings WA-BSB-01 to WA-BSB-03 advanced through up to 5.5 feet of very dense, brown, damp weathered dolostone bedrock. This soil unit has N-values of 50 blows per 3 inches to 50 blows per inch and a moisture content value of 4%.

5) Dolostone bedrock

At elevations of 584 to 593 feet (20.0 to 52.0 feet bgs), the borings encountered medium strong to strong, very poor to poor quality, slightly to moderately weathered dolostone bedrock. The rock quality designation (RQD) ranges from 0 to 41% and uniaxial compressive strength tests revealed Q_u values of 8,156 to 8,617 psi. Loss of coring water was noted in one of the boreholes. The bedrock core data are shown in the *Bedrock Core Photographs* (Appendix C).

3.2 Groundwater Conditions

Groundwater was encountered while drilling at elevations of 601 to 607 feet (6.3 to 36.8 feet bgs) within the silt to silty loam. For the purpose of analysis, the design groundwater elevation is considered at elevation 607 feet. It should be noted that groundwater levels might change with seasonal rainfall patterns and long-term climate fluctuations or may be influenced by local site conditions. Water levels seem consistent with rest of the project.

4.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS

Geotechnical evaluations and recommendations for the approach embankments, approach slabs, and substructure foundations are included in the following sections. The existing grade along the north and south approaches will be raised by up to 2.0 feet. This will require the placement of up to 2.0 feet of new fill behind the abutments. We assume the approach embankments will have side slopes graded at 1:2 (V: H), similar to the existing approach embankment side slopes.

Wang has evaluated possible foundation types for supporting the proposed bridge structure, and we recommend using driven deep foundations. Supporting the bridge substructures on shallow foundations is not feasible due to the large loads anticipated. Drilled shaft foundations are also not approved for use with integral abutments (IDOT 2020a); however, they could be considered at the pier.

4.1 Seismic Design Considerations

The seismic site class was determined in accordance with the IDOT *Geotechnical Manual* (IDOT 2020a). The soils within the top 100 feet have a weighted average N value of 91 blows/foot (Method C controlling), and the results classify the site in the Seismic Site Class C. The project location belongs to the Seismic Performance Zone 1 (IDOT 2020a). The seismic spectral acceleration parameters recommended for design in accordance with the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2020) are summarized in Table 1. According to the IDOT *Bridge Manual* (IDOT 2012), liquefaction analysis is not required for sites located in Seismic Performance Zone 1.

Table 1: Recommended Seismic Design Parameters

Spectral Acceleration Period (sec)	Spectral Acceleration Coefficient[1)] (% g)	Site Factors	Design Spectrum for Site Class C[2)] (% g)
0.0	PGA= 4.9	F_{pga}= 1.2	**A_s= 5.9**
0.2	S_s= 10.5	F_a= 1.2	**S_{DS}= 12.6**
1.0	S_1= 4.0	F_v= 1.7	**S_{D1}= 6.8**

1) Spectral acceleration coefficients based on Site Class C

2) Site Class C Spectrum to be included on plans; A_s = PGA*F_{pga}; S_{DS}= S_s*F_a; S_{D1}= S_1*F_v

4.2 Approach Embankments and Slabs

Wang has performed evaluations of the settlement and global stability of the approach embankments. The *Plan and Profile* drawings (Appendix E) show the proposed grade along Shepley Road will be raised by up to 2.0 feet to approximate pavement elevations of 638.8 to 637.6 feet at the north and south abutments, respectively. The grade along I-80 will be lowered by about 0.5 to 1.0 feet to an elevation of approximately 615.0 to 615.5 feet. We assume the approach embankments will have side slopes graded at 1:2 (V: H), similar to the existing approach embankment side slopes. Additionally, the *GPE* drawings (Appendix E) show concrete end slopes graded at 1:2 (V: H).

4.2.1 Settlement

To raise the grade along the approach embankments, up to 2.0 feet of new fill will be placed behind the abutments. Settlement estimates have been made based on correlations to measured index properties obtained from the laboratory tests (Appendix B). Based on the soil conditions, we estimate the foundation soils at the approaches will undergo up to 0.2 inch of long-term consolidation settlement under the applied load of the new approach embankment fill material. These settlements are appropriate for the construction of the approach slabs, and we do not anticipate downdrag allowances for the proposed abutment piles.

4.2.2 Global Stability

The global stability of the approach embankment side slopes was analyzed at the critical sections based on the soil profile described in Section 3.1 and the information provided in the plans. We also analyzed the stability of the end slope. The analysis discounts the beneficial effect of the abutment piles. The minimum required FOS for both short (undrained) and long-term (drained) conditions is 1.5 (IDOT 2012). *Slide2* evaluation exhibits employing the Bishop Simplified method of analysis are shown in Appendix D. The FOS values meet the minimum requirement.

4.2.3 Approach Slabs

We understand the approach slabs will be supported spread footing foundations (IDOT 2012). Based on the soil conditions revealed in Borings WA-BSB-01 and WA-BSB-02, the approach footings will be supported mainly on the new embankment fill resulting from the 2.0-foot high raise in grade along Wheeler Avenue. We estimate the fill has a maximum factored bearing resistance of 2,500 psf calculated for a geotechnical resistance factor (Φ_b) of 0.45 (AASHTO 2017). Settlement of the approach footing is not anticipated.

4.3 Structure Foundations

The foundation soil consist of stiff to hard clayey soils followed by dense to very dense silty loam and sandy gravel overlying dolostone bedrock. Wang recommends supporting the integral abutments on driven metal shell piles (MSP) or driven steel H-piles. The pier could be supported on either driven steel H-piles or drilled shafts.

The preliminary loading information provided by AECOM on August 31, 2021 is summarized in Table 2. As provided by AECOM, the proposed north and south abutment cap base elevation are 629.82 and 628.56 feet, respectively, whereas the proposed pier cap base elevation is 607.84 feet.

Table 2: Preliminary Factored Loads

Substructure	Total Factored Load (kips)
North Abutment	2400
Pier	6170
South Abutment	2270

4.3.1 Driven Piles

IDOT specifies the maximum nominal required bearing (R_{NMAX}) for each pile and states the factored resistance available (R_F) for steel H-piles and MSP should be based on a geotechnical resistance factor (ϕ_G) of 0.55 (IDOT 2012). Nominal tip and side resistance were estimated using the methods and empirical equations presented in the latest *IDOT Geotechnical Pile Design Guide* (IDOT 2020). Based on the loads provided by AECOM and the proposed width of the substructure, the load per pile at the abutments will range between about 122 and 343 kips for a single row of piles spaced at 3- to 8-feet. If piles are driven at the piers, the load per pile will range between 165 and 441 kips for two rows of piles spaced at 3- to 8-feet.

Based on IDOT standards, piles with greater than 0.4 inch of relative settlement along the sides require allowances for downdrag loads. We estimate that less than 0.4 inch of settlement will remain following the construction of the embankment and subsequent pile driving. We estimate that downdrag allowances will not be required for the abutment piles.

The foundation soils within 10.0 feet below the abutment pile cap elevations consist of very stiff to hard silty clay to silty clay loams with Q_u values of 3.7 to 8.7 tsf. In accordance with the *All*

Bridge Designers Memo 19.8 (IDOT 2019), when the average soil strengths at an integral abutment exceed 3.0 tsf, the piles at the abutments should be precored for a depth of 10.0 feet below the abutment cap elevation and backfilled with bentonite having a Q_u value of 1.0 tsf to increase pile flexibility (IDOT 2019). The pile capacity evaluations have been performed assuming pile driving begins about 10.0 feet below the proposed abutment pile cap elevations.

The R_F, R_N, estimated pile tip elevations, and pile lengths for 14-inch diameter MSP with 0.312-inch thick shells, 16-inch diameter MSP with 0.312-inch thick shells, HP12x53, and HP14x89 steel H-piles for the abutments are summarized in Tables 3 to 6. Table 7 provides pile lengths for HP14x89 and HP14x102 steel H-piles for the pier. The driving elevation was taken from the proposed cap elevations provided by AECOM. The pile lengths shown in Tables 3 to 6 assume a 2-foot pile embedment into the abutment pile cap and include the precored length of the pile, whereas the pile lengths shown in Table 7, assume a 1-foot embedment into the pier pile cap.

High blow counts, sampler refusal, and difficult drilling were noted within the borings below an approximate elevation of 599.0 feet indicating the presence of cobbles. As such, pile shoes should be used for piles driven below an elevation of 599 feet to avoid damage to the piles. Additionally, to achieve the maximum nominal required bearing at the north abutment and pier, the analysis shows the H-piles would need to be driven to the top of the bedrock at the north abutment and about 2 to 3 feet into the bedrock at the pier location. In these instances, the piles should be considered end bearing and designed for the maximum capacity of the pile. As per Section 6.13.2.4.2.2 of the IDOT *Geotechnical Manual*, when bedrock is within 10.0 to 15.0 feet below the bottom of a substructure, such as at the proposed pier location, soil-structure interaction analysis would need to be performed to determine if piles need to be set into rock. If the analysis indicates excessive pile head deflection, the pile would likely need to be set into rock to satisfy the deflection requirements (IDOT 2020). Battered piles could also be considered at the pier to satisfy lateral loading requirements (IDOT 2020).

Based on information provided by AECOM and the existing plans provided in the BCR, we understand the existing abutments and median pier may conflict with the driving of some of the piles. The existing structures should be removed in accordance with Section 501, *Removal of Existing Structures* of the IDOT *Standard Specifications* (IDOT 2016). The proposed abutment and pier pile locations should be selected to miss the existing footings.

Table 3: Estimated Pile Lengths and Tip Elevations for 14-inch Diameter MSP with 0.312-inch walls (Abutments)

Structure Unit (Reference Boring)	Pile Cap Base Elevations (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss (kips)	Factored Geotechnical Load Loss (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WA-BSB-01)	629.82	182	0	0	100	28	604
		273	0	0	150	31	601
		364	0	0	200	36	596
		455	0	0	250	37	595
		570(1)	0	0	314	38	594
South Abutment (WA-BSB-02)	628.56	182	0	0	100	30	601
		570(1)	0	0	314	31	600

(1) Maximum Nominal Required Bearing

Table 4: Estimated Pile Lengths and Tip Elevations for 16-inch Diameter MSP with 0.312-inch walls (Abutments)

Structure Unit (Reference Boring)	Pile Cap Base Elevations (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss (kips)	Factored Geotechnical Load Loss (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WA-BSB-01)	629.82	182	0	0	100	26	606
		273	0	0	150	30	602
		364	0	0	200	32	600
		455	0	0	250	36	596
		654(1)	0	0	360	37	595
South Abutment (WA-BSB-02)	628.56	182	0	0	100	28	603
		654(1)	0	0	360	32	599

(1) Maximum Nominal Required Bearing

Table 5: Estimated Pile Lengths and Tip Elevations for HP12x74 Steel Piles (Abutments)

Structure Unit (Reference Boring)	Pile Cap Base Elevations (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss (kips)	Factored Geotechnical Load Loss (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WA-BSB-01)	629.82	182	0	0	100	37	595
		273	0	0	150	39	593
		589(1)	0	0	324	40	592(2)
South Abutment (WA-BSB-02)	628.56	182	0	0	100	31	600
		273	0	0	150	34	597
		364	0	0	200	37	594
		455	0	0	250	41	590
		589(1)	0	0	324	45	586

(1) Maximum Nominal Required Bearing

(2) Approximate top of bedrock at Boring WA-BSB-01

Table 6: Estimated Pile Lengths and Tip Elevations for HP14x89 Steel Piles (Abutments)

Structure Unit (Reference Boring)	Pile Cap Base Elevations (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss (kips)	Factored Geotechnical Load Loss (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WA-BSB-01)	629.82	182	0	0	100	34	598
		273	0	0	150	39	593
		705(1)	0	0	388	40	592(2)
South Abutment (WA-BSB-02)	628.56	182	0	0	100	31	600
		273	0	0	150	33	598
		364	0	0	200	35	596
		455	0	0	250	38	593

Structure Unit (Reference Boring)	Pile Cap Base Elevations (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss (kips)	Factored Geotechnical Load Loss (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
		545	0	0	300	41	590
		636	0	0	350	43	588
		705(1)	0	0	388	44	587

(1) Maximum Nominal Required Bearing

(2) Approximate top of bedrock at Boring WA-BSB-01

Table 7: Estimated Pile Lengths and Tip Elevations at Pier

Reference Boring WA-BSB-03

Pile Cap Base Elevations (feet)	Pile Size	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss (kips)	Factored Geotechnical Load Loss (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
607.84	HP 14X89	705(1)	0	0	388	18	591(2)
	HP 14X102	811(1)	0	0	446	19	590(2)

(1) Maximum Nominal Required Bearing

(2) Piles driven 2 to 3 feet into the bedrock based on Boring WA-BSB-03 and pile length analysis (IDOT 2020).

4.3.2 Drilled Shafts

The pier could also be supported on drilled shafts established at the top of the bedrock, at an approximate elevation of 593.0 feet, or socketed into the bedrock.

As per 2012 IDOT Bridge Manual drilled shafts extending into rock, in most cases, should be designed utilizing only end bearing or side resistance in rock, whichever is larger. For shafts socketed into the bedrock less than 10-foot long, we estimate the end bearing will give more capacity than the side resistance. Therefore, we recommend considering only the end bearing resistance. The shafts should be designed for end bearing with a tip resistance factor (ϕ_{stat}) of 0.50 (AASHTO 2020). Above the bedrock, the shafts should have diameters 6 inches larger than the sockets.

The bedrock resistance was evaluated in accordance with the Geologic Strength Index (GSI) method provided by AASHTO (2020). The R_F, R_N, and estimated base elevations for shafts installed at the top of bedrock and rock-socketed shafts are summarized below in Tables 8 and 9. The lengths shown in the table include a 1-foot shaft embedment into the pier. For the anticipated loads (Table 2), we estimate shaft settlements of less than 0.5 inch.

Due to the presence of groundwater within the silt and silty loam soil and the presence of granular soils above the bedrock, the recommended construction method for shafts on top of bedrock and/or socketed into bedrock is to install casing to the top of the rock to maintain clean, open shafts during excavation. Since loss of water circulation was noted while coring and sinkholes are known to occur in this area, as discussed in Section 1.1, the quality of bedrock at the pier should be verified during construction.

Table 8: Estimated Drilled Shaft Resistances and Base Elevations (Top of Bedrock)

Structure Unit (Reference Boring)	Shaft Cap Base Elevations (feet)	Nominal Unit Resistance (ksf)	Estimated Shaft Length (feet)	Estimated Shaft Base Elevation (feet)	Shaft Diameter (feet)	Nominal Shaft Resistance, R_N (kips)	Factored Resistance Available, R_F (kips)
					3.0	1400	700
Pier (WA-BSB-03)	607.84	200	16.0	593(1)	40	2500	1250
					5.0	3900	1950

(1) Approximate top of bedrock at Boring WA-BSB-03

Table 9: Estimated Drilled Shaft Resistances and Base Elevations (Rock-Socketed Shafts)

Structure Unit (Reference Boring)	Shaft Cap Base Elevations (feet)	Top of Bedrock Elevation (feet)	Socket Diameter (feet)	Nominal Unit Resistance (ksf)	Nominal Resistance, R_N (kips)	Factored Resistance Available, R_F (kips)	Total Socket Length (feet)	Estimated Total Shaft Length (feet)
			3.0		2080	1040		
Pier (WA-BSB-03)	607.84	593.0	4.0	295	3700	1850	5.0	21.0
			5.0		5800	2900		

4.3.3 Lateral Loading

Lateral loads on the piles should be analyzed for maximum moments and lateral deflections. If the analysis at the proposed pier indicates excessive pile head deflection, the pile would need to be set into rock to satisfy the deflection requirements (IDOT 2020). Recommended lateral soil modulus and strain parameters required for analysis via the p-y curve method are included in Tables 10 to 13.

Table 10: Recommended Soil Parameters for Lateral Load Analysis at North Abutment
Reference Boring WA-BSB-01

Elevation Range (feet) Soil Type (Layer)	Unit Weight, γ (pcf)	Undrained Shear Strength, c_u (psf)	Estimated Friction Angle, Φ (°)	Estimated Lateral Soil Modulus Parameter, k (pci)	Estimated Soil Strain Parameter, ε_{50} (%)
629.8(1) to 619.8(2) New Fill (Bentonite)	120	1000	0	500	0.7
619.8 to 601 V Stiff to Hard SILTY CLAY to SILTY CLAY LOAM	120	3000	0	1000	0.5
601 to 598 M Dense SILTY LOAM	53(3)	0	30	60	--
598 to 594 M Dense to V Dense SANDY GRAVEL	58(3)	0	34	125	--
594 to 592.5(4) V Dense WEATHERED BEDROCK	58(3)	0	36	125	--

(1) Proposed pile cap base elevation;
(2) Approximate precoring length;
(3) Submerged unit weight;
(4) Approximate top of bedrock.

Table 11: Recommended Soil Parameters for Lateral Load Analysis at South Abutment

Reference Boring WA-BSB-02

Elevation Range (feet) Soil Type (Layer)	Unit Weight, γ (pcf)	Undrained Shear Strength, c_u (psf)	Estimated Friction Angle, Φ (°)	Estimated Lateral Soil Modulus Parameter, k (pci)	Estimated Soil Strain Parameter, ε_{50} (%)
628.6[1] to 618.6[2] New Fill (Bentonite)	120	1000	0	500	0.7
618.6 to 604 Stiff to V Stiff SILTY CLAY to SILTY CLAY LOAM	120	2500	0	1000	0.5
604 to 603 M Dense SILT	53[3]	0	30	60	--
603 to 599 V Stiff SILTY CLAY	58[3]	3100	0	1000	0.5
599 to 589 V Dense SANDY GRAVEL to SAND	58[3]	0	34	125	--
589 to 584[3] V Dense WEATHRERED BEDROCK	58[2]	0	36	125	--

(1) Proposed pile cap base elevation;

(2) Approximate precoring length;

(3) Submerged unit weight;

(4) Approximate top of bedrock.

Table 12: Recommended Soil Parameters for Lateral Load Analysis at Pier

Reference Boring WA-BSB-03

Elevation Range (feet) Soil Type (Layer)	Unit Weight, γ (pcf)	Undrained Shear Strength, c_u (psf)	Estimated Friction Angle, Φ (°)	Estimated Lateral Soil Modulus Parameter, k (pci)	Estimated Soil Strain Parameter, ε_{50} (%)
607.8[1] to 607 Stiff SILTY CLAY	120	1600	0	500	0.7
607 to 605 Loose SILT	53[2]	0	28	30	--
605 to 599 V Stiff SILTY CLAY	58[2]	3000	0	1000	0.5
599 to 598 M Dense SILT to SILTY LOAM	53[2]	0	30	60	--

Elevation Range (feet) Soil Type (Layer)	Unit Weight, γ (pcf)	Undrained Shear Strength, c_u (psf)	Estimated Friction Angle, Φ (°)	Estimated Lateral Soil Modulus Parameter, k (pci)	Estimated Soil Strain Parameter, ε_{50} (%)
598 to 595 V Dense SAND	58(2)	0	34	125	--
595 to 593(3) V Dense WEATHERED BEDROCK	58(2)	0	36	125	--

(1) Proposed pile cap base elevation;

(2) Submerged unit weight;

(3) Approximate top of bedrock.

Table 13: Recommended Bedrock Parameters for Lateral Load Analysis

Reference Boring WA-BSB-01 to WA-BSB-03

Bedrock	Total Unit Weight, γ (pcf)	Modulus of Rock Mass (ksi)	Uniaxial Compressive Strength (psi)	RQD (%)	Strain Factor
Dolostone	140	300	8,000 (Estimated)	0 to 10	0.0005
Dolostone	140	1000	8,156 and 8,617	26 to 41	0.0005

4.4 Stage Construction

Based on the GPE, Wang understands the bridge replacement will be performed under a road closure and detour of Wheeler Avenue and stage construction will not be required.

Excavations of up to 9.0 feet below the existing grade along Wheeler Avenue will be required for the construction of the abutments. In addition, the removal and replacement of the pier in the median of I-80 will require up to 5.0 feet of excavation. Temporary support systems will be required if the ground cannot be sloped at 1:2 (V: H). We estimate temporary steel sheet piling, designed using the charts included in the *IDOT Design Guide-Simplified Temporary Sheet Piling Design Charts* is feasible (IDOT 2020a).

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Site Preparation

Vegetation, surface topsoil, and debris should be cleared and stripped where the structure will be placed. If unstable or unsuitable materials are exposed during excavation, they should be removed and replaced with compacted structural fill as described in Section 5.3.

5.2 Excavation, Dewatering, and Utilities

Excavations should be performed in accordance with local, state, and federal regulations. The potential effect of ground movements upon nearby utilities should be considered during construction. Any slope that cannot be graded at 1:2 (V:H) should be properly shored in accordance with the temporary sheet piling charts provided in *IDOT Design Guide-Simplified Temporary Sheet Piling Design Charts* (IDOT 2020).

During the subsurface investigation, the groundwater was encountered at elevations ranging from 607 to 601 feet, as discussed in Section 3.2. At the north and south abutments, the groundwater will be about 24.0 to 29.0 feet below the pile cap base elevations, respectively; therefore, we do not anticipate the need for dewatering. At the pier excavation the groundwater will be less than 1.0 foot below the pile cap elevation and dewatering efforts should be anticipated at this location. Perched, or temporary water, may be encountered during times of heavy precipitation while excavating within the upper fill soils and will require dewatering efforts. Water that does accumulate in open excavations by seepage or runoff should be immediately removed by sump pump.

5.3 Filling and Backfilling

Fill material used to attain final design elevations should be pre-approved, compacted, cohesive or granular soil conforming to Section 204, *Borrow and Furnished Excavation* (IDOT 2016). The fill material should be free of organic matter and debris and should be placed in lifts and compacted according to Section 205, *Embankment* (IDOT 2016).

Backfill materials for the abutments and piers must be pre-approved by the Resident Engineer. To backfill the abutments, we recommend porous granular material conforming to the requirements specified in the IDOT Supplemental Special and Recurring Special Provisions, *Granular Backfill for Structures* (IDOT 2020b).

5.4 Earthwork Operations

The required earthwork can be accomplished with conventional construction equipment. Moisture and traffic will cause deterioration of exposed subgrade soils. Precautions should be taken by the Contractor to prevent water erosion of the exposed subgrade. A compacted subgrade will minimize water runoff erosion.

Earth moving operations should be scheduled to not coincide with excessive cold or wet weather (early spring, late fall or winter). Any soil allowed to freeze or soften due to the standing water should be removed. Wet weather can cause problems with subgrade compaction.

It is recommended that an experienced geotechnical engineer be retained to inspect the exposed subgrade, monitor earthwork operations, and provide material inspection services during the construction phase of this project.

5.5 Pile Installation

The driven piles shall be furnished and installed according to the requirements of IDOT Section 512, *Piling* (IDOT 2016). Wang recommends performing one test pile at each substructure location. Since hard driving is expected below an elevation of 599 feet, pile shoes are required as indicated in Section 4.3.1.

5.6 Drilled Shaft Construction

Drilled shafts should be constructed in accordance with IDOT Section 516, *Drilled Shafts* (IDOT 2016). Control of the groundwater will be of high importance during the excavation of shafts. We recommend installing casing to the top of the rock to maintain clean, open shafts during excavation.

Hard drilling conditions were encountered in the borings below an elevation of 599 feet, indicating the presence of cobbles. The potential of encountering cobbles and/or boulders during shaft excavation should be taken into consideration by the Contractor.

6.0 QUALIFICATIONS

The analysis and recommendations submitted in this report are based upon the data obtained from the borings drilled at the locations shown on the boring logs and in Exhibit 3. This report does not reflect any variations that may occur between the borings or elsewhere on the site, variations whose nature and extent may not become evident until the course of construction. In the event that any changes in the design and/or location of the structure are planned, we should be timely informed so that our recommendations can be adjusted accordingly.

It has been a pleasure to assist TranSystems Corporation, AECOM, and the Illinois Department of Transportation on this project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

WANG ENGINEERING, INC.

Azza Hamad, P.E.
Senior Geotechnical Engineer

Nesam Balakumaran, P.Eng.
Project Geotechnical Engineer

Liviu Iordache, P.G.
QC/QA Reviewer

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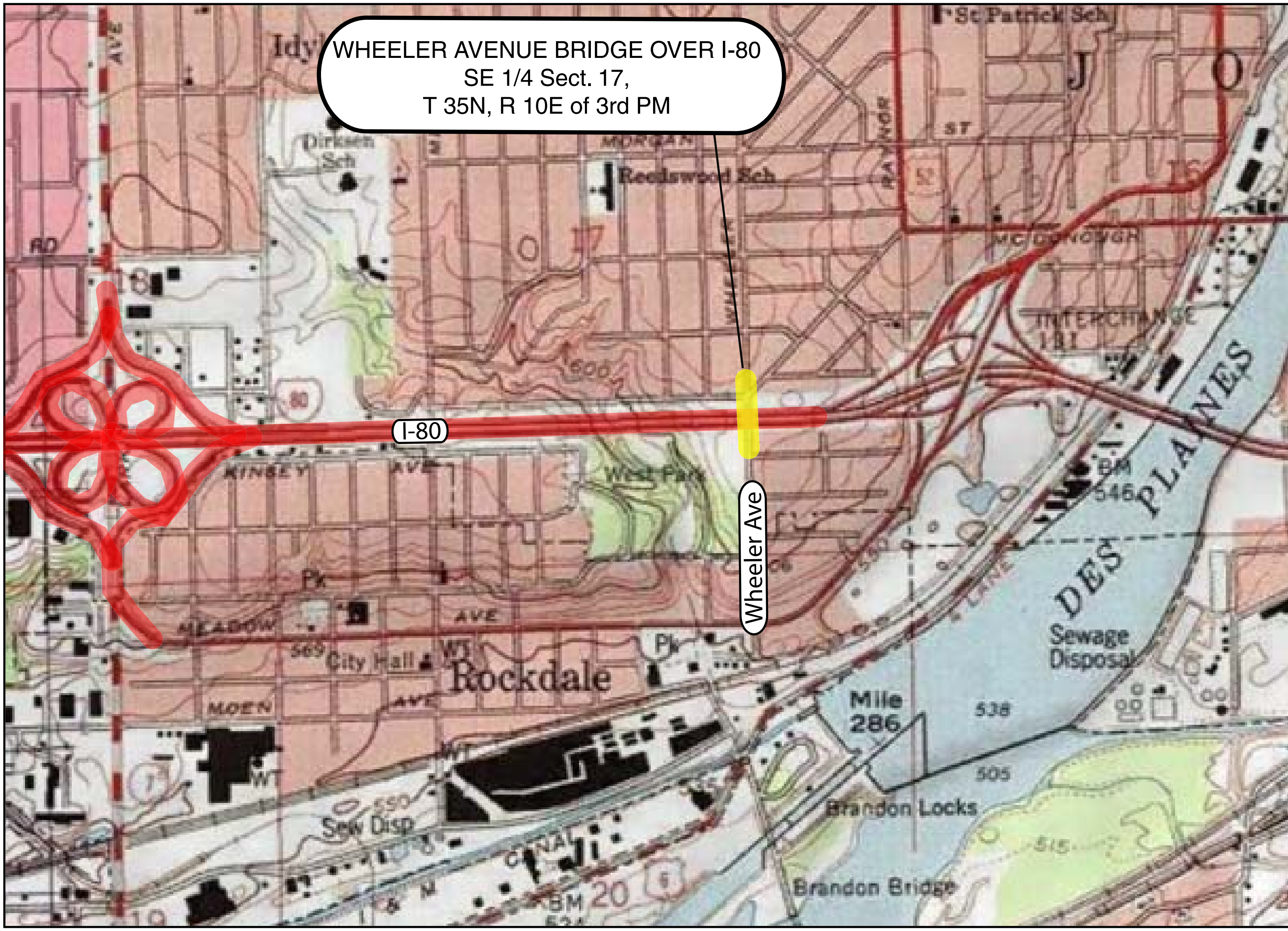
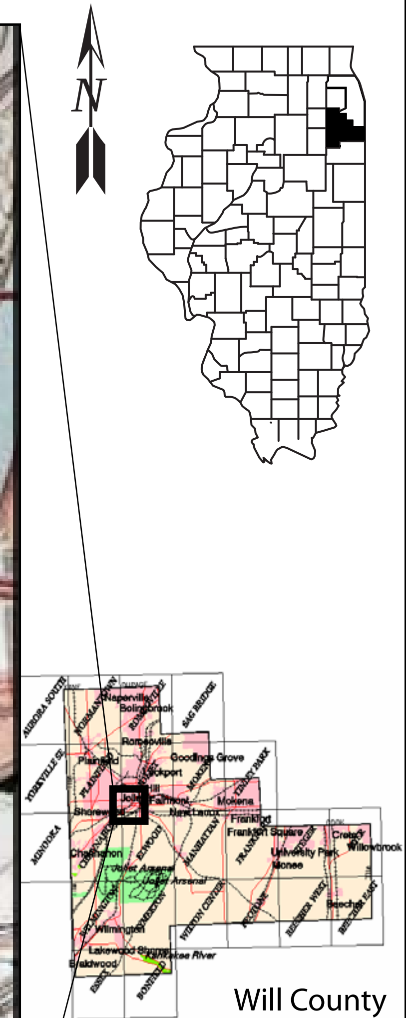
EXHIBITS





0 0.5 1.0 Mile

SITE LOCATION MAP: WHEELER AVENUE BRIDGE OVER I-80 ; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET & LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL	EXHIBIT 1	DRAWN BY: J. Bensen CHECKED BY: A. Hamad

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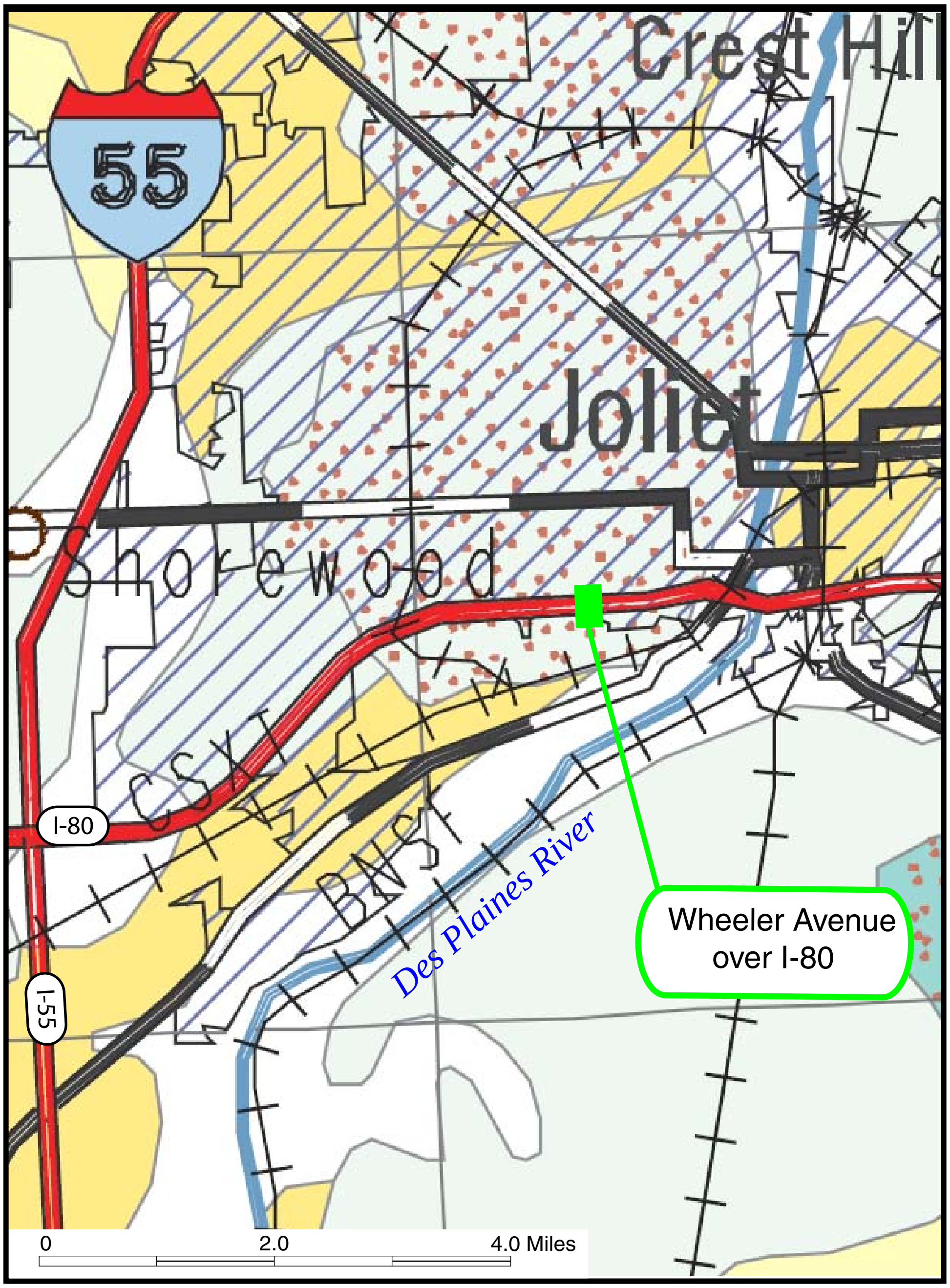
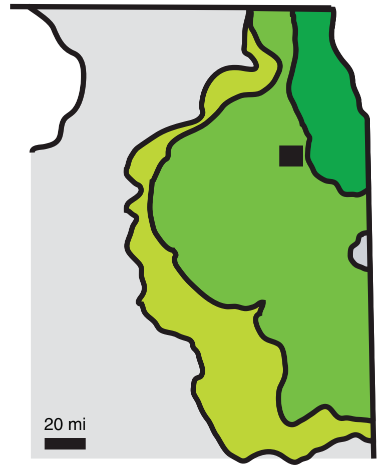
7901-15-01



Modified after William W. Shilts (2000)

N

REGIONAL GEOLOGY



Wedron Group

- Wadsworth Formation
- Lemont Formation
- Tiskilwa Formation

Modified after Hansel and Johnson (1996)

Legend

Cahokia Formation
Bedded silts, clays and sand and gravel deposited in floodplains and channels of modern rivers and streams

Equality Formation
Laminated silt, clay, or fine sand deposited in slackwater, glacial, and postglacial lakes; massive to bedded, calcareous

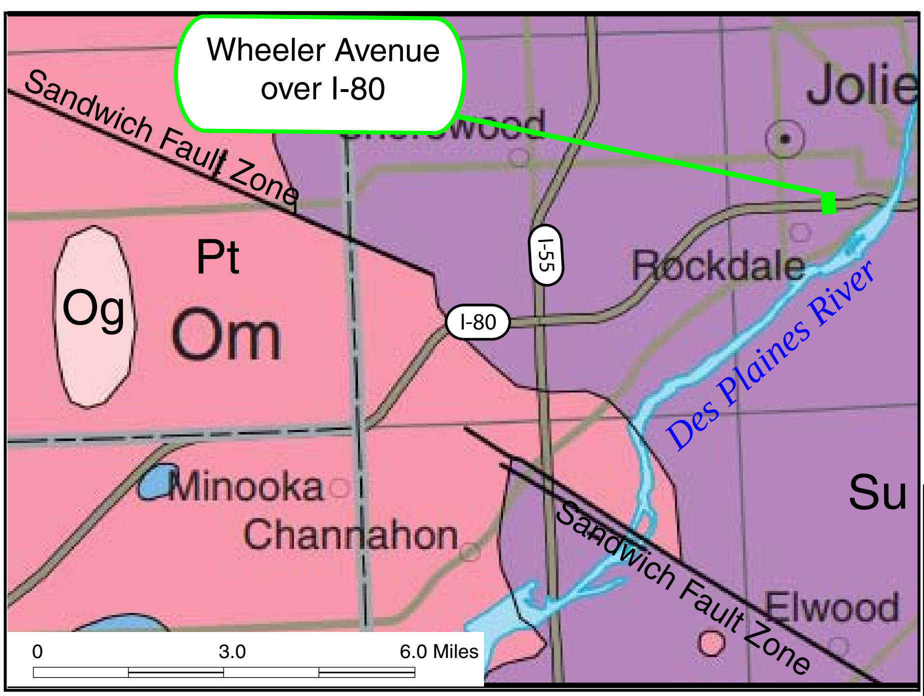
Henry Formation
Predominantly stratified sand and gravel; locally occurring silt, clay, organic debris, wood, shell lenses; outwash on floodplains

Lemont Formation
Diamicton; silty clay loam to silty clay; fine grained matrix; unsorted, unstratified mixture of gravel, sand, silt, and clay; containing lenses of sand, gravel and/or silt.

Bedrock near or at the surface

Major metropolitan area

Modified after Shilts (2000)



Modified after Dennis R. Kolata (2005)

Legend

- Pt — **Pennsylvanian Shale** Tadewater Formation
- Su — **Silurian Dolomitic Limestone**
- Om — **Ordovician Formation** Maquoketa Group
- Og — **Ordovician Formation** Galena Group

SITE AND REGIONAL GEOLOGY: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL	EXHIBIT 2	DRAWN BY: C. Marin CHECKED BY: L. Iordache

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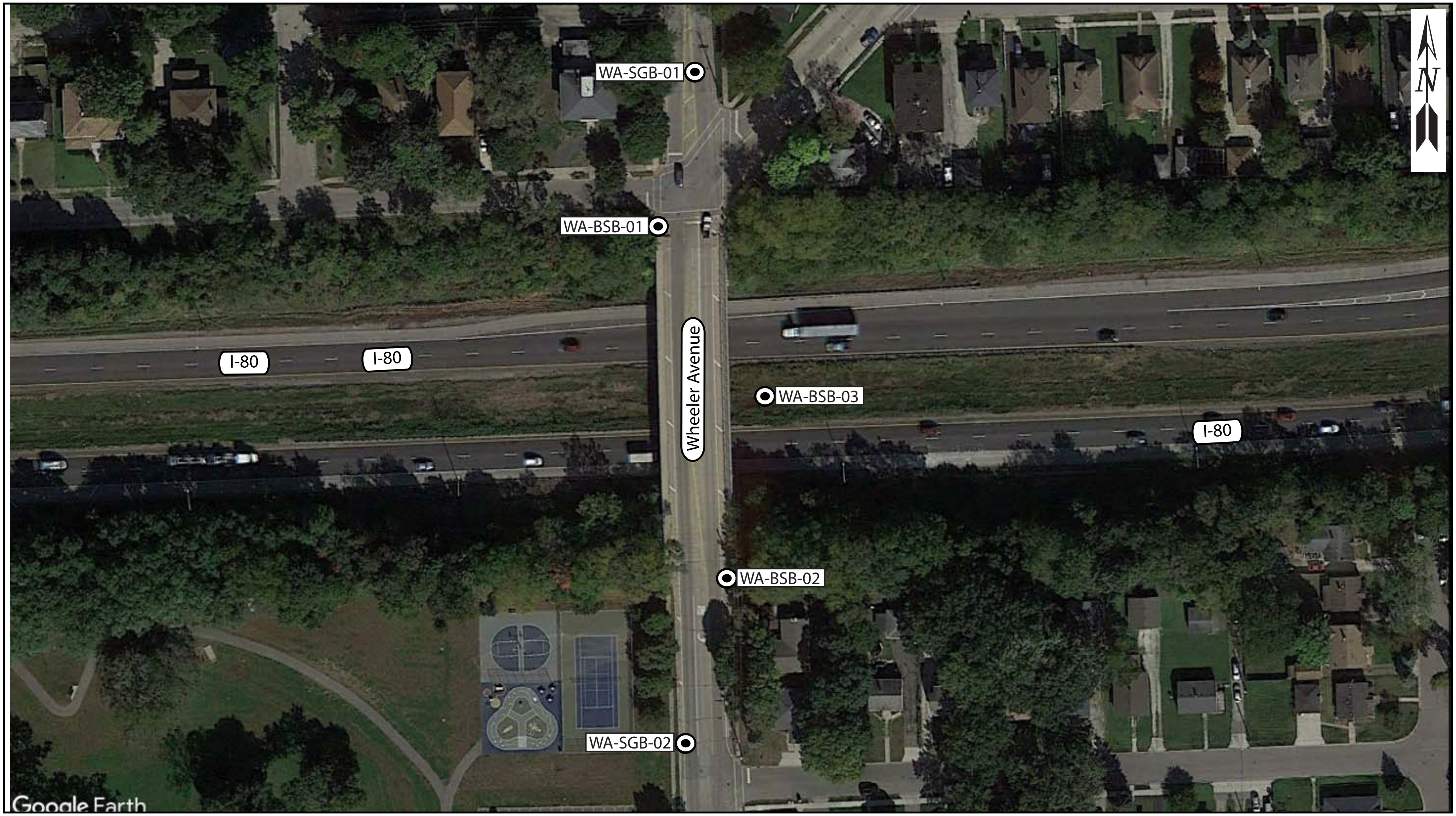

0 200 400 Feet

Legend

Boring Location

BORING LOCATION PLAN: WHEELER AVENUE OVER I-80 BRIDGE; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS		
SCALE: GRAPHICAL	EXHIBIT 3	DRAWN BY: J. Bensen CHECKED BY: A. Hamad
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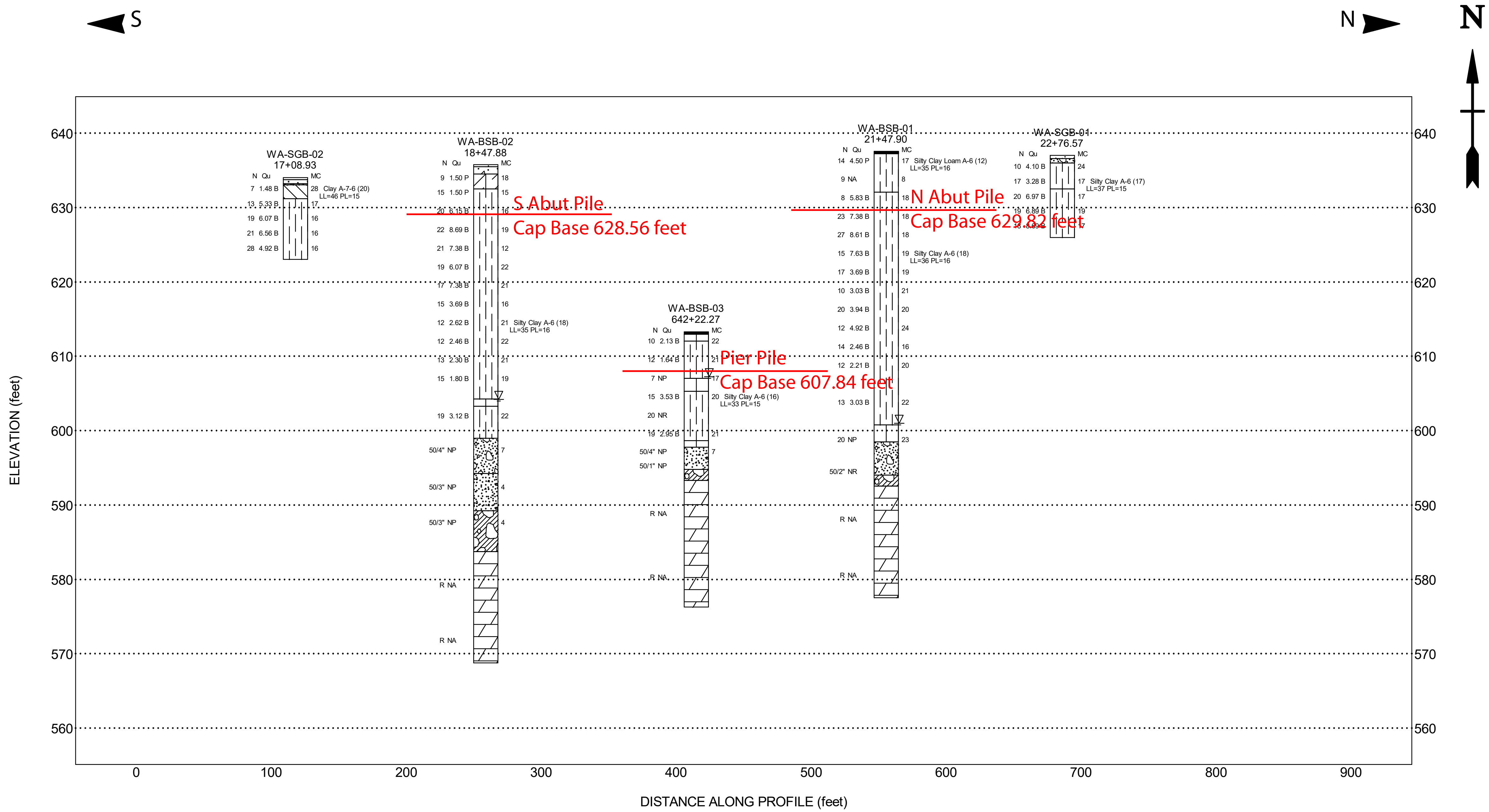
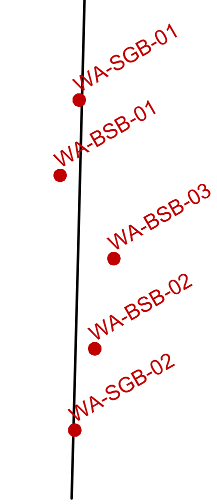


Lithology Graphics

- Topsoil
- Weathered bedrock
- IDH Clay Loam
- IDH Silty Clay, Silty Clay Loam
- Dolomite or Dolomitic Limestone
- Coarse sand
- IDH Silt, Silty Loam
- Pavement
- IDH Clay
- Gravelly sand, sandy gravel
- Concrete



Site Map Scale 1 inch equals 330 feet

Explanation:

WA-BSB-01
21+47.90 — Borehole Number
Station

Borehole Lithology

N--N-value, (blw/12 in)
Qu--UC Strength, (tsf)
MC--Moisture Content, (%)

Water Level Reading at time of drilling.

Water Level Reading 24-hr after drilling or at end of drilling

0 90

Horizontal Scale (feet)

Vertical Exaggeration: 5.5x

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Soil Profile
Wheeler Avenue over Interstate 80
PR SN 099-8324

WEI
SINCE 1982

I-80 Reconstruction (Houbolt Road to Center Street)
Will County, Illinois

JOB NUMBER	PLATE NUMBER
7901-15-01	EXHIBIT 4

WEI 11X17 79011501.GPJ WANGENG.GDT 9/9/21

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APPENDIX A

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BORING LOG WA-BSB-01

WEI Job No.: 7901-15-01

Client: **TranSystems Corporation**
Project: **I-80 Reconstruction (Houbolt Road to Center Street)**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 637.53 ft
North: 1764933.78 ft
East: 1046708.69 ft
Station: 21+47.90
Offset: 32.52 LT

SOIL AND ROCK DESCRIPTION

Elevation (ft)	Description
637.2	4-inch thick, black SILTY CLAY LOAM --TOPSOIL--
	Hard, brown SILTY CLAY LOAM, trace to some gravel; damp --FILL-- --RDR 2-- --L_L(%)=35, P_L(%)=16-- --%Gravel=10.8-- --%Sand=14.4-- --%Silt=48.1-- --%Clay=26.7-- --A-6 (12)--
632.0	Very stiff to hard, brown to gray SILTY CLAY to SILTY CLAY LOAM, trace gravel; damp --RDR 2-- --L_L(%)=36, P_L(%)=16-- --%Gravel=2.4-- --%Sand=5.9-- --%Silt=56.3-- --%Clay=35.4-- --A-6 (18)--
600.8	Gray SILTY LOAM; wet --RDR 2--
598.5	Medium dense to very dense, brown SANDY GRAVEL; damp

Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
1	4, 7, 7	4.50 P	17
2	11, 4, 5	NA	8
3	3, 3, 5	5.83 B	18
4	9, 10, 13	7.38 B	18
5	11, 12, 15	8.61 B	18
6	4, 7, 8	7.63 B	19
7	5, 8, 9	3.69 B	19
8	4, 4, 6	3.03 B	21
9	6, 9, 11	3.94 B	20
10	5, 6, 6	4.92 B	24
11	5, 6, 8	2.46 B	16
12	5, 5, 7	2.21 B	20
13	5, 5, 8	3.03 B	22
14	6, 2, 18	NP	23

Depth (ft): 5, 10, 15, 20, 25, 30, 35, 40

GENERAL NOTES

Begin Drilling **03-23-2021** Complete Drilling **03-23-2021**
Drilling Contractor **Wang Testing Services** Drill Rig **20CME55T[81%]**
Driller **RR&J** Logger **I. Nenn** Checked by **C. Marin**
Drilling Method **2.25" IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling ∇ **36.75 ft**
At Completion of Drilling ▼ **NA**
Time After Drilling **NA**
Depth to Water ▼ **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG WA-BSB-01

WEI Job No.: 7901-15-01

Client: **TranSystems Corporation**
Project: **I-80 Reconstruction (Houbolt Road to Center Street)**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 637.53 ft
North: 1764933.78 ft
East: 1046708.69 ft
Station: 21+47.90
Offset: 32.52 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type recovery	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	--RDR 2-5--						
	--possible cobbles--						
594.0	--difficult drilling from 43.5 feet-- --possible cobbles-- --WEATHERED BEDROCK--			15	50/2"	NR	
592.5	Strong, light gray, very poor to poor quality, DOLOSTONE; very closely to closely spaced, moderately weathered, horizontal joints, with <0.05 inch opening, rough walls, and <0.2 inch thick clay infill. --RUN 1: 45.0 to 55.0 feet-- --Recovery = 100%-- --RQD= 9%--	45			CORE		
		50		16			
	--RUN 2: 55.0 to 60.0 feet-- --Recovery = 100%-- --RQD= 41%-- --Q_u= 8,156 psi--	55			CORE		
				17			
577.5	Boring terminated at 60.00 ft	60					

GENERAL NOTES

Begin Drilling **03-23-2021** Complete Drilling **03-23-2021**
Drilling Contractor **Wang Testing Services** Drill Rig **20CME55T[81%]**
Driller **RR&J** Logger **I. Nenn** Checked by **C. Marin**
Drilling Method **2.25" IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling **36.75 ft**
At Completion of Drilling **NA**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG WA-BSB-02

WEI Job No.: 7901-15-01

Client: **TranSystems Corporation**
Project: **I-80 Reconstruction (Houbolt Road to Center Street)**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 635.74 ft
North: 1764635.42 ft
East: 1046768.31 ft
Station: 18+47.88
Offset: 18.04 RT

Elevation (ft)	Soil and Rock Description	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
635.5	3-inch thick ASPHALT --PAVEMENT--					
634.5	12-inch thick CONCRETE --PAVEMENT--					
	Stiff, black and gray SILTY CLAY to CLAY LOAM, trace gravel; damp --FILL-- --RDR 2--		1	10, 5, 4	1.50 P	18
632.5	Stiff to hard, brown to gray SILTY CLAY to SILTY CLAY LOAM, trace gravel; damp to moist --RDR 2--	5	2	3, 6, 9	1.50 P	15
			3	6, 9, 11	6.15 B	16
		10	4	6, 9, 13	8.69 B	19
			5	6, 9, 12	7.38 B	12
		15	6	6, 8, 11	6.07 B	22
			7	6, 7, 10	7.38 B	21
		20	8	6, 6, 9	3.69 B	16
	--L_L(%)=35, P_L(%)=16-- --%Gravel=1.1-- --%Sand=3.3-- --%Silt=58.3-- --%Clay=37.4-- --A-6 (18)--		9	4, 6, 6	2.62 B	21
		25	10	5, 5, 7	2.46 B	22
			11	4, 6, 7	2.30 B	21
		30	12	4, 6, 9	1.80 B	19
604.2	Possible SILT; saturated --wet spoon recovery--					
603.2	Very stiff, gray SILTY CLAY; damp --RDR 2--	35	13	5, 7, 12	3.12 B	22
599.0	Very dense, brown SANDY GRAVEL; damp --RDR 2-- --possible cobbles--		14	37, 50/4"	NP	7
		40				

GENERAL NOTES

Begin Drilling: **03-22-2021** Complete Drilling: **03-22-2021**
Drilling Contractor: **Wang Testing Services** Drill Rig: **20CME55T[81%]**
Driller: **RR&J** Logger: **I. Nenn** Checked by: **C. Marin**
Drilling Method: **3.25" IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling: **31.75 ft**
At Completion of Drilling: **NA**
Time After Drilling: **NA**
Depth to Water: **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG WA-BSB-02

WEI Job No.: 7901-15-01

Client: **TranSystems Corporation**
Project: **I-80 Reconstruction (Houbolt Road to Center Street)**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 635.74 ft
North: 1764635.42 ft
East: 1046768.31 ft
Station: 18+47.88
Offset: 18.04 RT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type (recovery)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
594.2	Very dense, brown, coarse SAND; damp --RDR 2-3-- --possible cobbles--			15	35 50/3"	NP	4
		45					
589.2	Very dense, brown SANDY GRAVEL; damp --WEATHERED BEDROCK-- --possible cobbles--			16	50/3"	NP	4
		50					
583.7	Medium strong to strong, light grayish gray, very poor to poor quality, DOLOSTONE; very closely spaced, slightly to moderately weathered, horizontal, oblique, and vertical joints, with <0.05 inch opening, rough walls, and no infill. --RUN 1: 52.0 to 62.0 feet-- --Recovery = 93%-- --RQD= 6%--	55		17	CORE		
		60					
	--RUN 2: 62.0 to 67.0 feet-- --Recovery = 100%-- --RQD= 35%--	65		18	CORE		
568.7	Boring terminated at 67.00 ft						
		70					
		75					
		80					

GENERAL NOTES

Begin Drilling: **03-22-2021** Complete Drilling: **03-22-2021**
Drilling Contractor: **Wang Testing Services** Drill Rig: **20CME55T[81%]**
Driller: **RR&J** Logger: **I. Nenn** Checked by: **C. Marin**
Drilling Method: **3.25" IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling: **31.75 ft**
At Completion of Drilling: **NA**
Time After Drilling: **NA**
Depth to Water: **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG WA-BSB-03

WEI Job No.: 7901-15-01

Client: **TranSystems Corporation**
Project: **I-80 Reconstruction (Houbolt Road to Center Street)**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 613.28 ft
North: 1764790.60 ft
East: 1046801.06 ft
Station: 642+22.27
Offset: 1.46 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
613.0	4-inch thick, black SILTY CLAY LOAM --TOPSOIL--					
612.0	Gray SILTY LOAM; damp					
	Medium stiff (0.75P) to very stiff, gray SILTY CLAY, trace gravel; damp to moist --RDR 2--		1	3 4 6	2.13 B	22
		5	2	5 5 7	1.64 B	21
607.0	Loose, brown and gray SILT, few sand lenses; saturated --RDR 2--		3	4 3 4	NP	17
605.3	Very stiff, gray SILTY CLAY, trace gravel; damp --RDR 2-- --L_L(%)=33, P_L(%)=15-- --%Gravel=2.3-- --%Sand=6.3-- --%Silt=53.6-- --%Clay=37.8-- --A-6 (16)--	10	4	2 6 9	3.53 B	20
			5	8 10 10	NR	
		15	6	4 6 13	2.95 B	21
598.6	Gray SILT to SILTY LOAM; saturated					
597.8	Very dense, brown, medium to coarse SAND, trace gravel; damp --RDR 2-3--		7	23 30 50/4"	NP	7
594.8	--frequent rig chatter at 18.5 feet-- --possible cobbles--		8	50/1"	NP	
593.3		20				
	--WEATHERED BEDROCK-- Strong, light gray, very poor to poor quality, DOLOSTONE; very closely to closely spaced, slightly weathered, horizontal, oblique, and vertical joints, with 0 inch opening, slightly rough walls, and no infill. --RUN 1: 20.0 to 30.0 feet-- --Recovery = 90%-- --RQD= 0%-- --no water return from 21.5 to 37.0 feet bgs--	25	9	CORE		
	--RUN 2: 30.0 to 37.0 feet-- --Recovery = 96%-- --RQD= 26%-- --Q_u=8,617 psi	30, 35	10	CORE		
576.3	Boring terminated at 37.00 ft	40				

GENERAL NOTES

Begin Drilling: **04-30-2021** Complete Drilling: **04-30-2021**
Drilling Contractor: **Wang Testing Services** Drill Rig: **20D25A [83%]**
Driller: **NC&KG** Logger: **E. Yim** Checked by: **C. Marin**
Drilling Method: **2.25" IDA HSA to 10 ft; mud rotary thereafter; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling: **6.25 ft**
At Completion of Drilling: **mud in borehole**
Time After Drilling: **NA**
Depth to Water: **NA**

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BORING LOG WA-SGB-01

WEI Job No.: 7901-15-01

Client: **TranSystems Corporation**
Project: **I-80 Reconstruction (Houbolt Road to Center Street)**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 636.98 ft
North: 1765063.51 ft
East: 1046741.61 ft
Station: 22+76.57
Offset: 4.31 RT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
636.6	5-inch thick ASPHALT --PAVEMENT--					
636.0	7-inch thick, gray SANDY GRAVEL --BASE COURSE--		1	2 4 6 7	4.10 B	24
	Very stiff to hard, gray SILTY CLAY, trace gravel; damp --FILL-- --RDR 2-- --L_L(%)=37, P_L(%)=15-- --%Gravel=4.1-- --%Sand=12.5-- --%Silt=52.1-- --%Clay=31.3-- --A-6 (17)--		2	6 7 10 14	3.28 B	17
632.5	Hard, brown SILTY CLAY, trace gravel; damp --RDR 2--	5	3	7 9 11 12	6.97 B	17
			4	8 9 10 13	6.89 B	19
		10	5	5 8 10 13	8.69 B	17
626.0	Boring terminated at 11.00 ft					
		15				
		20				

GENERAL NOTES

Begin Drilling: **03-23-2021** Complete Drilling: **03-23-2021**
Drilling Contractor: **Wang Testing Services** Drill Rig: **20CME55T[81%]**
Driller: **RR&J** Logger: **I. Nenn** Checked by: **C. Marin**
Drilling Method: **2.25" IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling: **DRY**
At Completion of Drilling: **DRY**
Time After Drilling: **NA**
Depth to Water: **NA**

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BORING LOG WA-SGB-02

WEI Job No.: 7901-15-01

Client: **TranSystems Corporation**
Project: **I-80 Reconstruction (Houbolt Road to Center Street)**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 634.01 ft
North: 1764495.37 ft
East: 1046733.89 ft
Station: 17+08.93
Offset: 20.61 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
633.8	3-inch thick ASPHALT --PAVEMENT--					
633.2	7-inch thick CONCRETE --PAVEMENT--					
633.0	2-inch thick, gray SANDY GRAVEL; damp --BASE COURSE--		1	2, 3, 4, 5	1.48 B	28
631.2	Stiff, brown CLAY, trace to few gravel; damp --FILL-- --RDR 2-- --L_L(%)=46, P_L(%)=15-- --%Gravel=8.8-- --%Sand=19.6-- --%Silt=41.8-- --%Clay=29.7-- --A-7-6 (20)--	5	2	6, 5, 8, 9	5.33 B	17
			3	6, 8, 11, 12	6.07 B	16
	Hard, brown SILTY CLAY, trace gravel; damp --RDR 2--		4	7, 10, 11, 15	6.56 B	16
		10	5	11, 14, 14, 14	4.92 B	16
623.0	Boring terminated at 11.00 ft	15, 20				

GENERAL NOTES

Begin Drilling	**03-22-2021**
Complete Drilling	**03-22-2021**
Drilling Contractor	**Wang Testing Services**
Drill Rig	**20CME55T[81%]**
Driller	**RR&J**
Logger	**I. Nenn**
Checked by	**C. Marin**
Drilling Method	**2.25" IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling	**DRY**
At Completion of Drilling	**DRY**
Time After Drilling	**NA**
Depth to Water	**NA**

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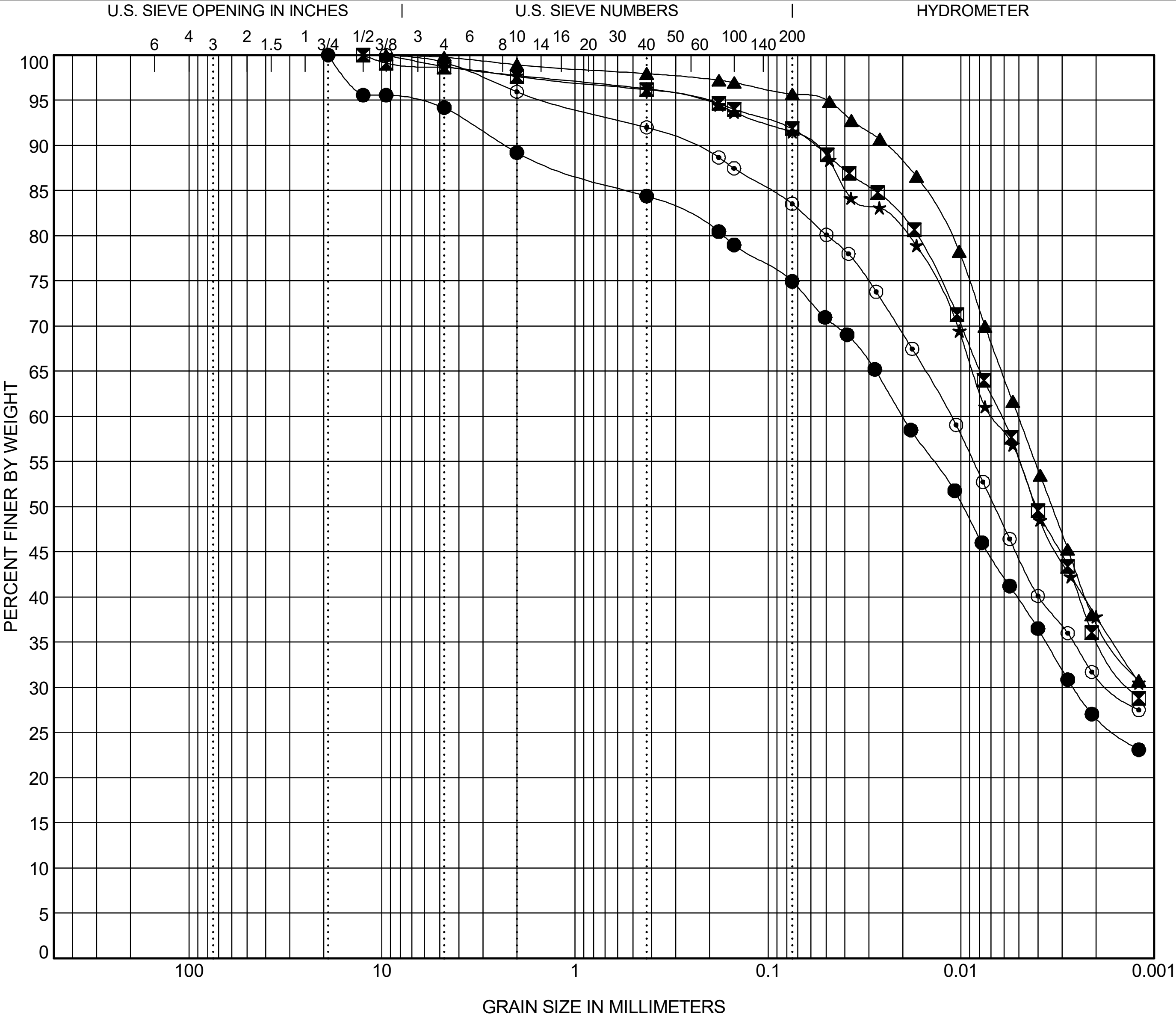
APPENDIX B



COBBLES	GRAVEL	SAND		SILT AND CLAY
		coarse	fine	

Specimen Identification		IDH Classification	LL	PL	PI	Cc	Cu
● WA-BSB-01#1	1.0 ft	**Silty Clay Loam**	35	16	19		
⊠ WA-BSB-01#6	13.5 ft	**Silty Clay**	36	16	20		
▲ WA-BSB-02#9	21.0 ft	**Silty Clay**	35	16	19		
★ WA-BSB-03#4	8.5 ft	**Silty Clay**	33	15	18		
⊙ WA-SGB-01#2	3.0 ft	**Silty Clay**	37	15	22		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● WA-BSB-01#1	1.0 ft	19	0.02	0.003		10.8	14.4	48.1	26.7
⊠ WA-BSB-01#6	13.5 ft	12.5	0.006	0.001		2.4	5.9	56.3	35.4
▲ WA-BSB-02#9	21.0 ft	9.5	0.005			1.1	3.3	58.3	37.4
★ WA-BSB-03#4	8.5 ft	9.5	0.007			2.3	6.3	53.6	37.8
⊙ WA-SGB-01#2	3.0 ft	9.5	0.011	0.002		4.1	12.5	52.1	31.3

WEI
SINCE 1982

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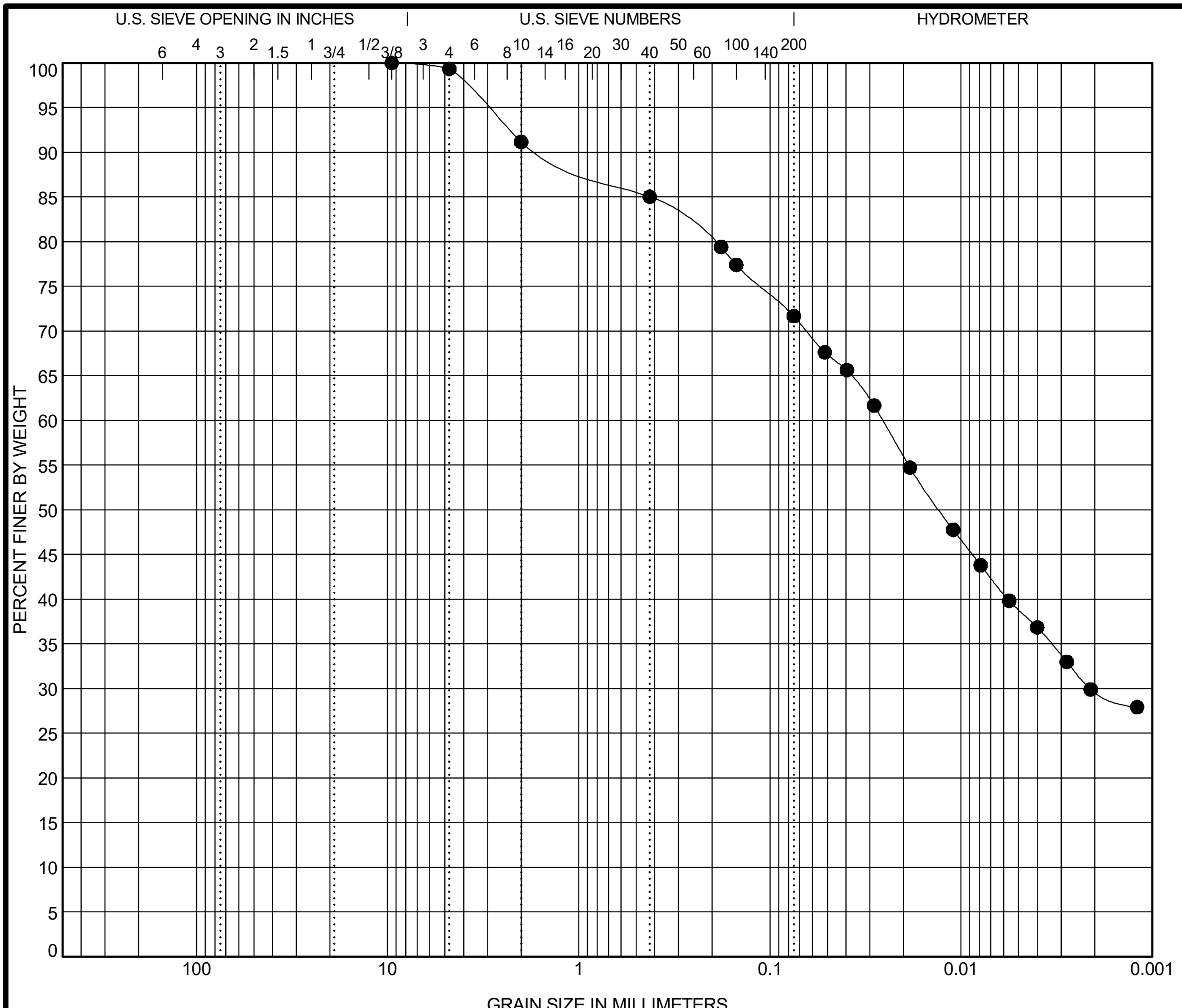
GRAIN SIZE DISTRIBUTION

Project: I-80 Reconstruction (Houbolt Road to Center Street)

Location: Will County, Illinois

Number: 7901-15-01



COBBLES	GRAVEL	SAND		SILT AND CLAY
		coarse	fine	

	Specimen Identification		IDH Classification	LL	PL	PI	Cc	Cu
●	**WA-SGB-02#1**	**1.0 ft**	**Clay**	**46**	**15**	**31**		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**WA-SGB-02#1**	**1.0 ft**	**9.5**	**0.026**	**0.002**		**8.8**	**19.6**	**41.8**	**29.7**

WEI GRAIN SIZE IDH 79011501.GPJ US LAB.GDT 9/13/21

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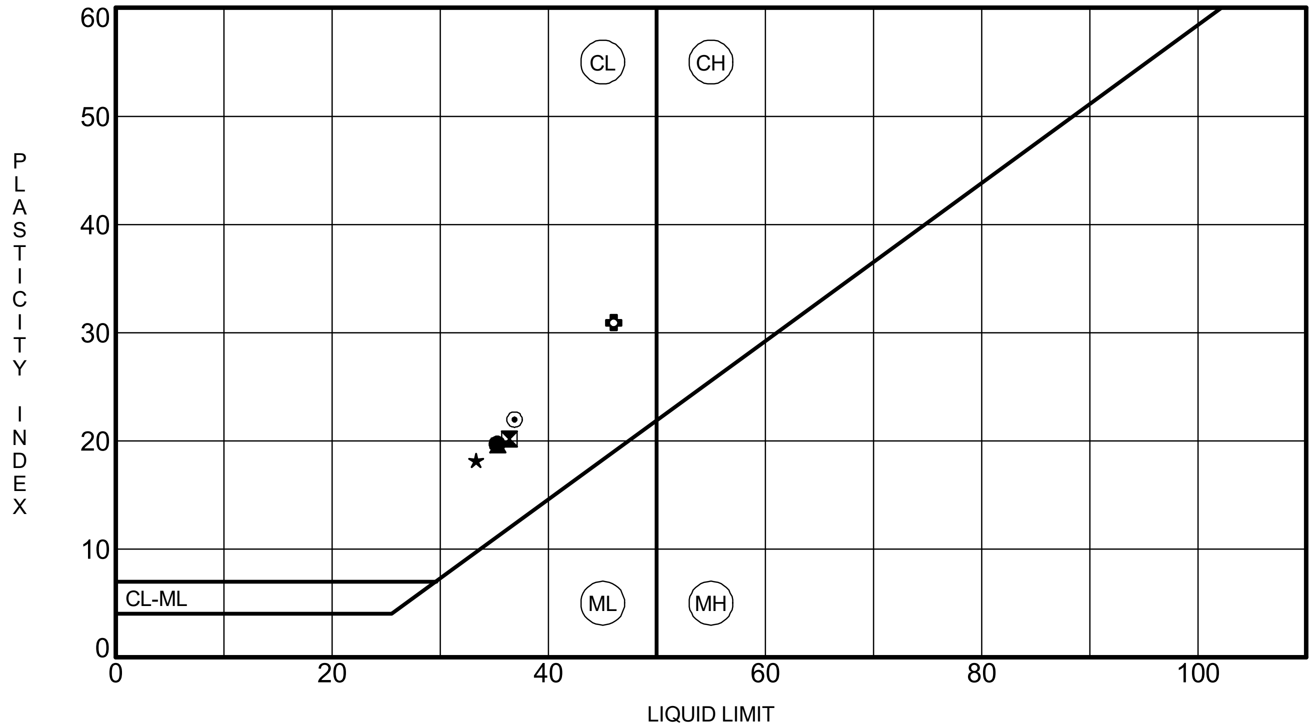
GRAIN SIZE DISTRIBUTION

Project: I-80 Reconstruction (Houbolt Road to Center Street)

Location: Will County, Illinois

Number: 7901-15-01



	Specimen Identification		LL	PL	PI	Fines	IDH Classification
●	**WA-BSB-01#1**	**1.0 ft**	**35**	**16**	**19**	**75**	**Silty Clay Loam**
⧗	**WA-BSB-01#6**	**13.5 ft**	**36**	**16**	**20**	**92**	**Silty Clay**
▲	**WA-BSB-02#9**	**21.0 ft**	**35**	**16**	**19**	**96**	**Silty Clay**
★	**WA-BSB-03#4**	**8.5 ft**	**33**	**15**	**18**	**92**	**Silty Clay**
⊙	**WA-SGB-01#2**	**3.0 ft**	**37**	**15**	**22**	**84**	**Silty Clay**
✿	**WA-SGB-02#1**	**1.0 ft**	**46**	**15**	**31**	**72**	**Clay**

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ATTERBERG LIMITS' RESULTS

Project: I-80 Reconstruction (Houbolt Road to Center Street)

Location: Will County, Illinois

Number: 7901-15-01

Unconfined Compressive Strength of Intact Rock Core Specimens

Project: I-80

Client: Transystems

WEI Job No.: 7901-15-01

Field Sample ID	Run #	Depth (ft)	Location	Sample Description	Length (in) Before Capping	Length (in) After Capping	Diameter (in)	Total Load (lbs)	Total Pressure (psi)	Fracture Type*	Break Date	Tested By	Area (in^2)
WA-BSB-01	1	49.5	North Abutment	Dolostone	4.00	NA	2.05	26920	8156	3	5/26/21	MAC	3.30
WA-BSB-03	2	33.0	Pier	Dolostone	4.20	NA	2.06	28720	8617.1	3	9/7/21	MAC	3.33

*** Fracture Types:**

Type 1 - Reasonably well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps;
Type 2 - Well-formed cone on one end, vertical cracks running through caps, no well defined cone on other end;
Type 3 - Columnar vertical cracking through both ends, no well-formed cones;
Type 4 - Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type 1;
Type 5 - Side fractures at top or bottom (occur commonly with unbonded caps);
Type 6 - Similar to Type 5 but end of cylinder is pointed.

Prepared by:______________________________

Checked by: ______________________________

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APPENDIX C

Run #1

0 6 inches

Boring WA-BSB-01:
Run #1, 45.0 to 55.0 feet, RECOVERY=100%, RQD=9%

BEDROCK CORE: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS		
SCALE: GRAPHICAL	APPENDIX C-1	DRAWN BY: J. Bensen CHECKED BY: A. Hamad
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR TRANSYSTEMS CORPORATION		7901-15-01

Run #2

Boring WA-BSB-01:
Run #2, 55.0 to 60.0 feet, RECOVERY=100%, RQD=41%

BEDROCK CORE: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS		
SCALE: GRAPHICAL	APPENDIX C-2	DRAWN BY: J. Bensen CHECKED BY: A. Hamad
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FOR TRANSYSTEMS CORPORATION		7901-15-01

Run #1

0 6 inches

Boring WA-BSB-02:
Run #1, 52.0 to 62.0 feet, RECOVERY=93%, RQD=6%

BEDROCK CORE: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS		
SCALE: GRAPHICAL	APPENDIX C-3	DRAWN BY: J. Bensen CHECKED BY: A. Hamad
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FOR TRANSYSTEMS CORPORATION		7901-15-01

Run #2

0 6 inches

Boring WA-BSB-02:
Run #2, 62.0 to 67.0 feet, RECOVERY=100%, RQD=35%

BEDROCK CORE: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS		
SCALE: GRAPHICAL	APPENDIX C-4	DRAWN BY: J. Bensen CHECKED BY: A. Hamad
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FOR TRANSYSTEMS CORPORATION		7901-15-01

Run #1

Boring WA-BSB-03:
Run #1, 20.0 to 30.0 feet, RECOVERY=90%, RQD=0%

BEDROCK CORE: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS		
SCALE: GRAPHICAL	APPENDIX C-5	DRAWN BY: J. Bensen CHECKED BY: A. Hamad
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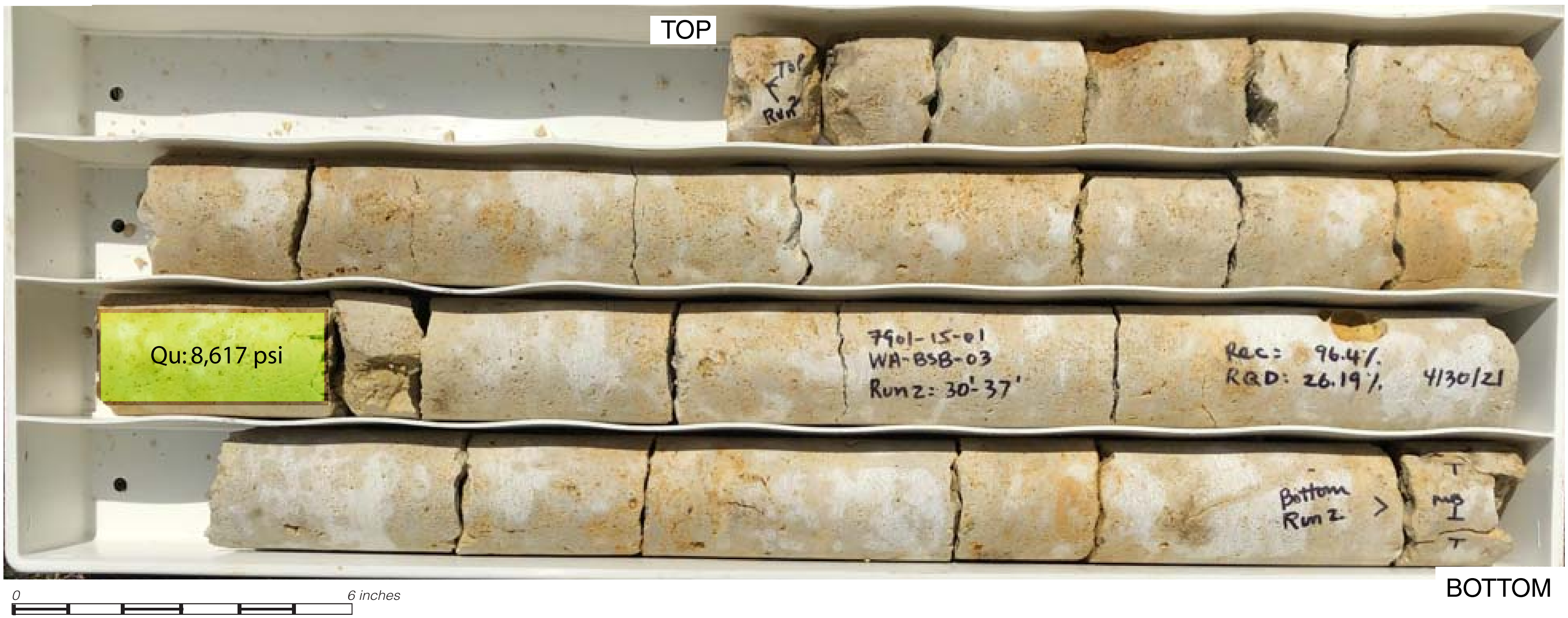
FOR TRANSYSTEMS CORPORATION		7901-15-01

Run #2



Boring WA-BSB-03:
Run #2, 30.0 to 37.0 feet, RECOVERY=96%, RQD=26%

BEDROCK CORE: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS		
SCALE: GRAPHICAL	APPENDIX C-6	DRAWN BY: J. Bensen CHECKED BY: A. Hamad
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FOR TRANSYSTEMS CORPORATION		7901-15-01

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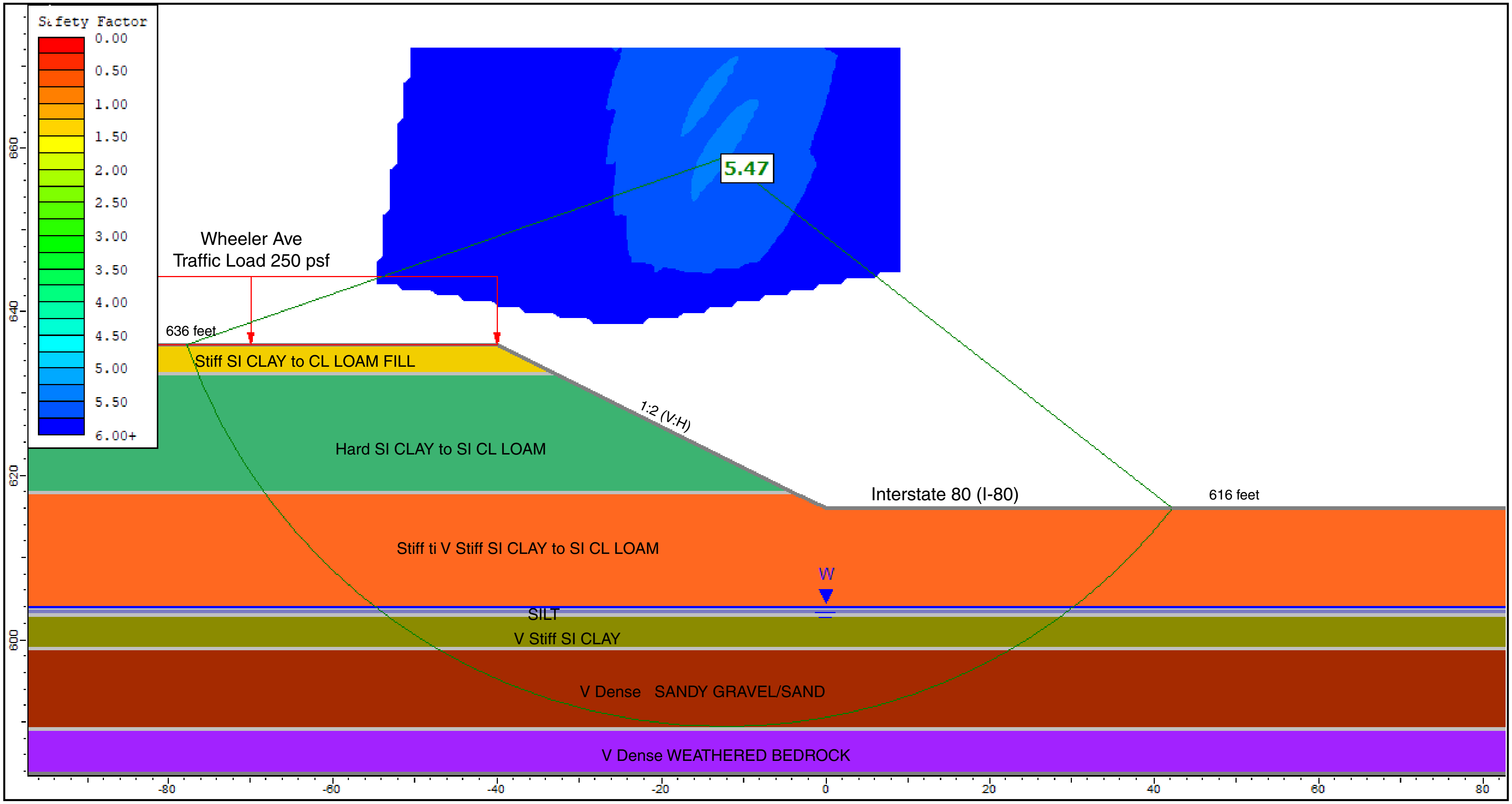
APPENDIX D



Undrained Analysis, South Abutment Side Slope, Reference Boring: WA-BSB-02

Layer ID	Description	Total Unit Weight (pcf)	Undrained Cohesion (psf)	Undrained Friction Angle (degrees)
1	Stiff SI CLAY to CL LOAM Fill	125	1500	0
2	Hard SI CLAY to SI CL LOAM	125	4000	0
3	Stiff to V Stiff SI CLAY to SI CL LOAM	120	2500	0
4	SILT	120	0	30
5	V Stiff SI CLAY	125	3100	0
6	V Dense SANDY GRAVEL/SAND	125	0	34
7	V Dense WEATHERED BEDROCK	130	0	36

GLOBAL STABILITY: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL

APPENDIX D-1

DRAWN BY: N. Balakumaran
CHECKED BY: A. Hamad

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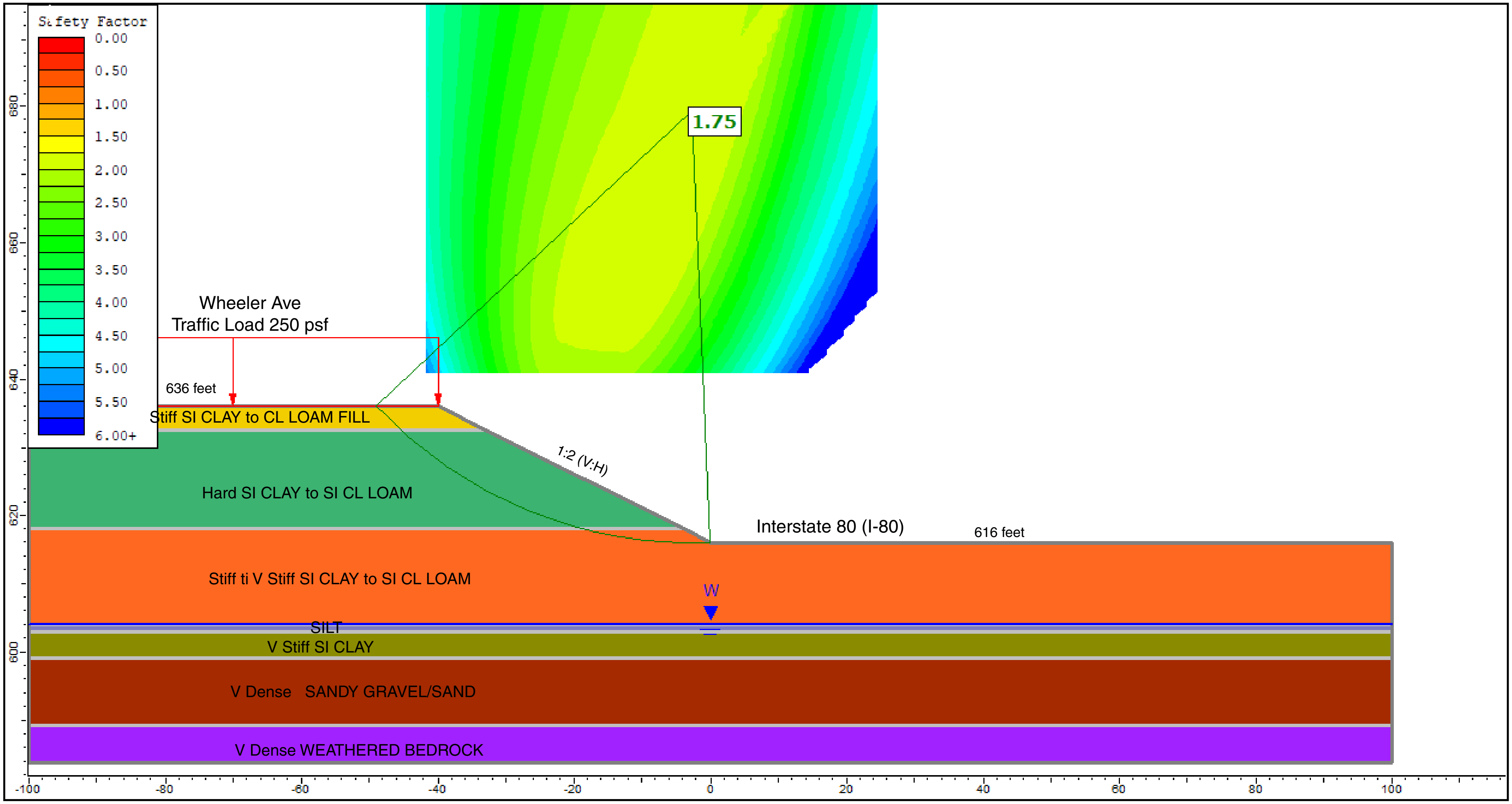
FOR TRANSYSTEMS CORPORATION

7901-15-01



Drained Analysis, South Abutment Side Slope, Reference Boring: WA-BSB-02

Layer ID	Description	Total Unit Weight (pcf)	Drained Cohesion (psf)	Drained Friction Angle (degrees)
1	Stiff SI CLAY to CL LOAM Fill	125	100	30
2	Hard SI CLAY to SI CL LOAM	125	100	30
3	Stiff to V Stiff SI CLAY to SI CL LOAM	120	100	30
4	SILT	120	0	30
5	V Stiff SI CLAY	125	100	30
6	V Dense SANDY GRAVEL/SAND	125	0	34
7	V Dense WEATHERED BEDROCK	130	0	36

GLOBAL STABILITY: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL	APPENDIX D-2	DRAWN BY: N. Balakumaran CHECKED BY: A. Hamad

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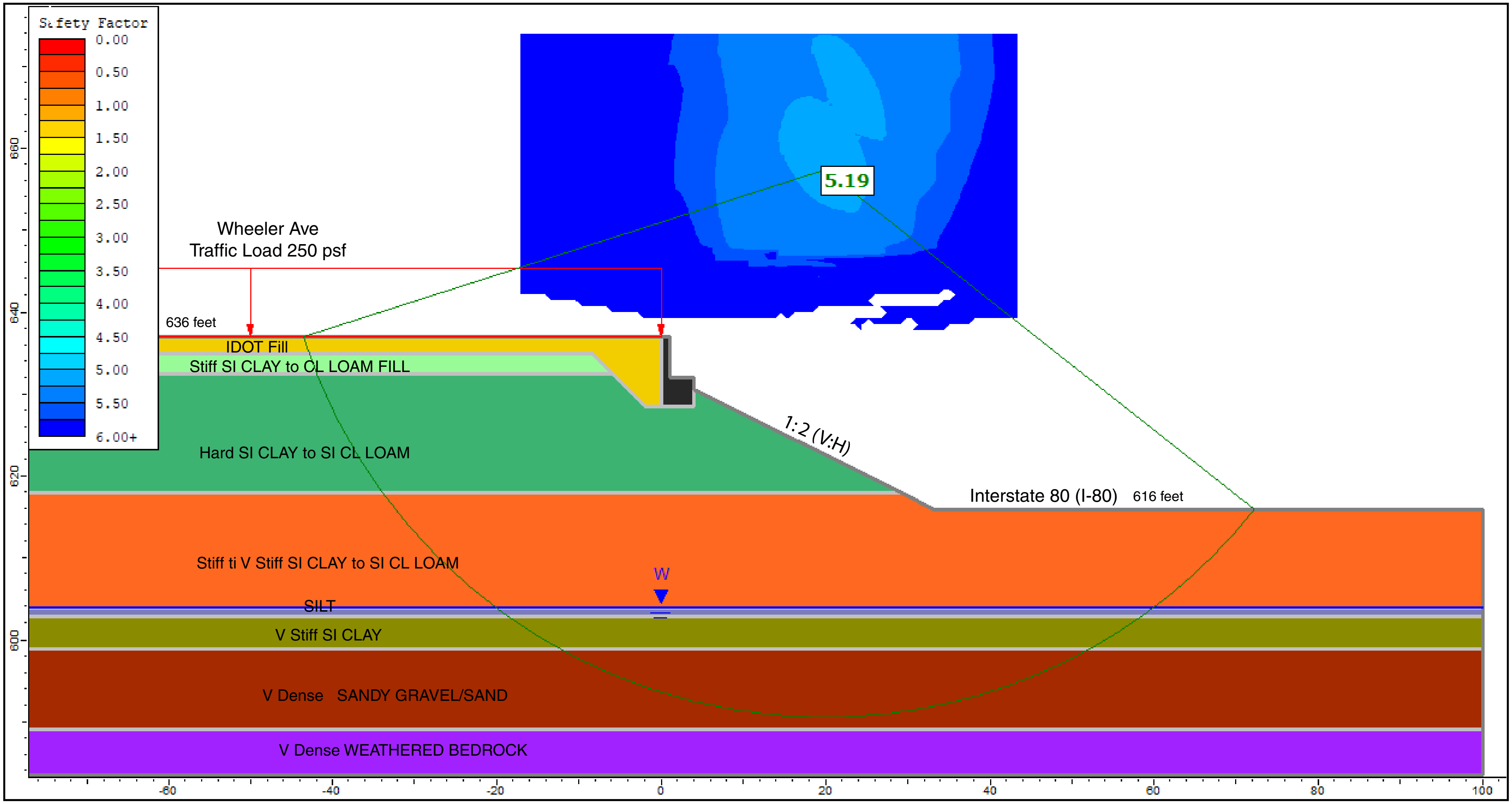
7901-15-01



Undrained Analysis, South Abutment End Slope, Reference Boring: WA-BSB-02

Layer ID	Description	Total Unit Weight (pcf)	Undrained Cohesion (psf)	Undrained Friction Angle (degrees)
1	IDOT FILL	125	1000	0
2	Stiff SI CLAY to CL LOAM Fill	120	1500	0
3	Hard SI CLAY to SI CL LOAM	125	4000	0
4	Stiff to V Stiff SI CLAY to SI CL LOAM	120	2500	0
5	SILT	120	0	30
6	V Stiff SI CLAY	125	3100	0
7	V Dense SANDY GRAVEL/SAND	125	0	34

GLOBAL STABILITY: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL

APPENDIX D-3

DRAWN BY: N. Balakumaran
CHECKED BY: A. Hamad

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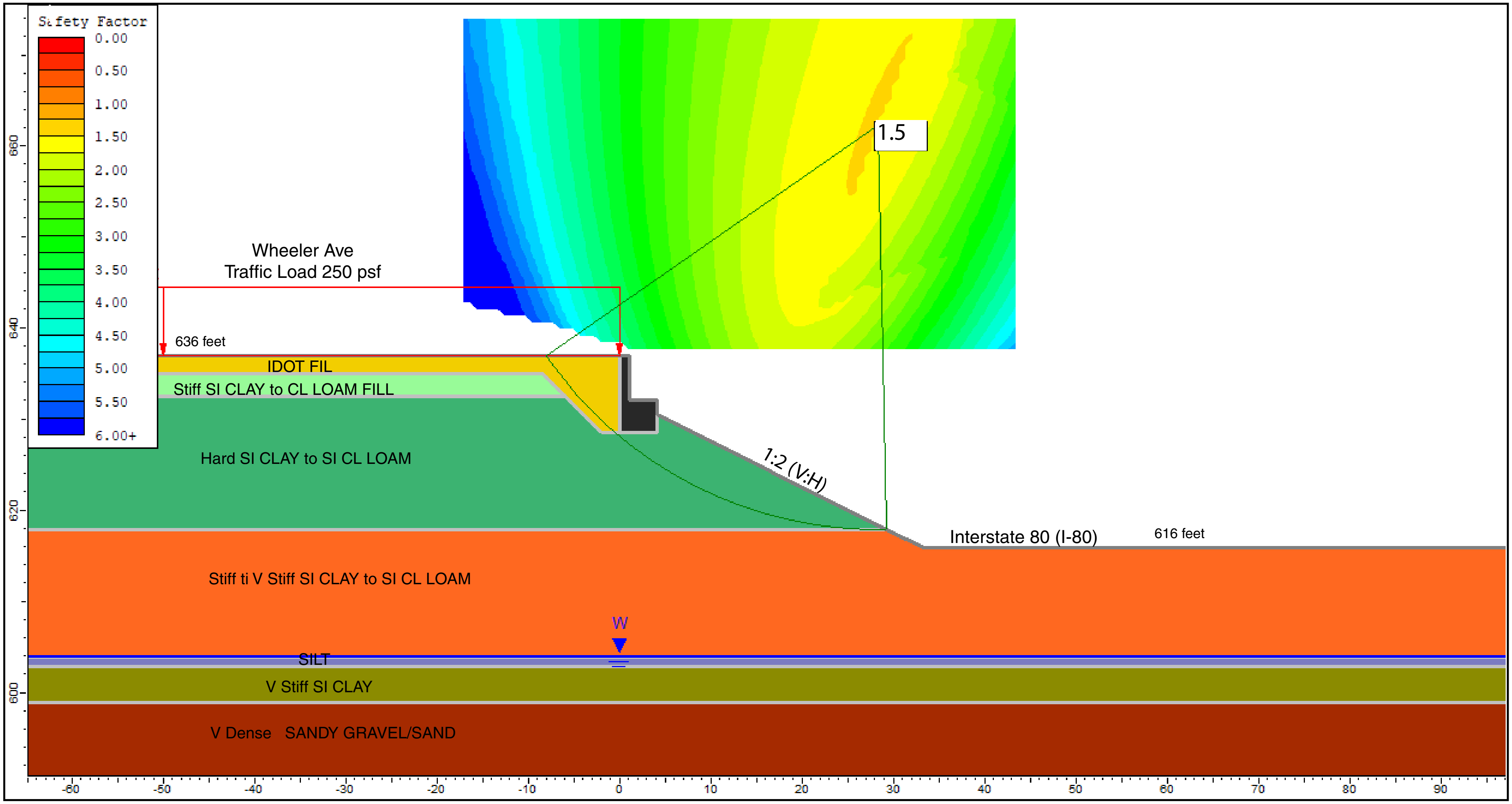
7901-15-01



Drained Analysis, South Abutment End Slope, Reference Boring: WA-BSB-02

Layer ID	Description	Total Unit Weight (pcf)	Drained Cohesion (psf)	Drained Friction Angle (degrees)
1	IDOT FILL	120	100	30
2	Stiff SI CLAY to CL LOAM Fill	125	100	30
3	Hard SI CLAY to SI CL LOAM	125	100	30
4	Stiff to V Stiff SI CLAY to SI CL LOAM	120	100	30
5	SILT	120	0	30
6	V Stiff SI CLAY	125	100	30
7	V Dense SANDY GRAVEL/SAND	125	0	34

GLOBAL STABILITY: WHEELER AVENUE BRIDGE OVER I-80; I-80 RECONSTRUCTION FROM HOUBOLT ROAD TO WEST OF CENTER STREET AND LARKIN AVENUE INTERCHANGE, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL

APPENDIX D-4

DRAWN BY: N. Balakumaran
CHECKED BY: A. Hamad

Wang Engineering
1145 N. Main Street
Lombard, IL 60148
www.wangeng.com

7901-15-01

1145 North Main Street
Lombard, Illinois 60148
Phone (630) 953-9928
www.wangeng.com

APPENDIX E

Benchmark: Set cut square in northwesterly corner of southerly overhead sign foundation for sign "East Indiana", "Exit 131 Center St" on the south side of east bound I-80 @ Center Street exit ramp; Elev. 619.468.

Existing Structure: SN 099-0175, built in 1966 as FAI Rte. 80, Project I-80-4 (22) 133, Section 99-3. The structure has a total of 4 spans with continuous steel stringers that support the deck. The original substructure consisted of cast-in-place reinforced concrete stub abutments and multiple column grade separation piers supported on spread footings. The structure was repaired in 2002 under Project ACBHI-80-4(180)133, Section 99-3HB-I. The improvement consisted of bridge deck replacement, superstructure and substructure rehabilitation, addition of shear studs to steel beams, and approach roadway reconstruction. The abutments were converted to a semi-integral type abutment. The bridge measures 237'-6" back to back abutments, 60'-0" out to out width with no skew at abutments and a skew of 1°04'42" at piers. The structure will be replaced under a road closure.

No Salvage.

HIGHWAY CLASSIFICATION

FAI Rte. 80 - I-80	FAU 340 - Wheeler Ave.
Functional Class: Interstate	Functional Class: Minor Collector
ADT: 75,800 (2019); 115,326 (2032)	ADT: 1,950 (2019); 2,749 (2032)
ADTT: 12,886 (2019); 19,606 (2032)	ADTT: 137 (2019); 193 (2032)
DHV: -,--- (2032)	DHV: --- (2032)
Design Speed: 70 m.p.h.	Design Speed: 30 m.p.h.
Posted Speed: 65 m.p.h.	Posted Speed: 25 m.p.h.
Two-Way Traffic	Two-Way Traffic
Directional Distribution: 50:50	Directional Distribution: 50:50

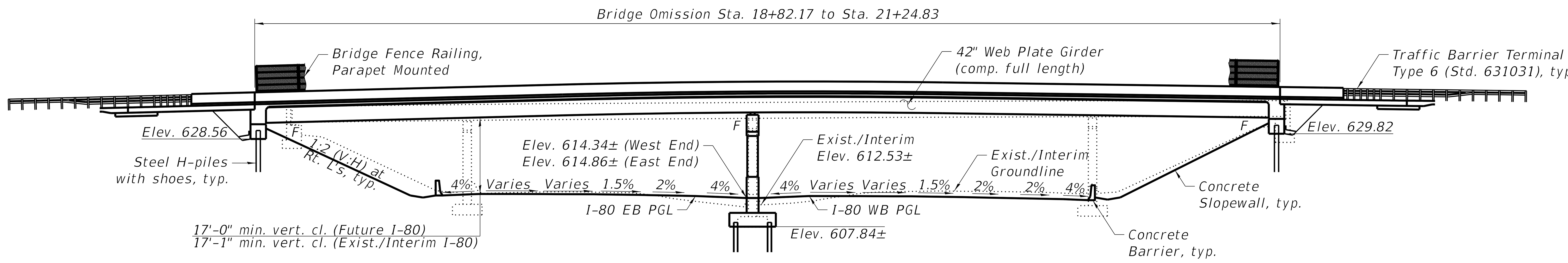
DESIGN SPECIFICATIONS

2020 AASHTO LRFD Bridge Design Specifications, 9th Edition

DESIGN STRESSES

FIELD UNITS

f'c = 3,500 psi
f'c = 4,000 psi (Superstructure)
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (M270 Grade 50)



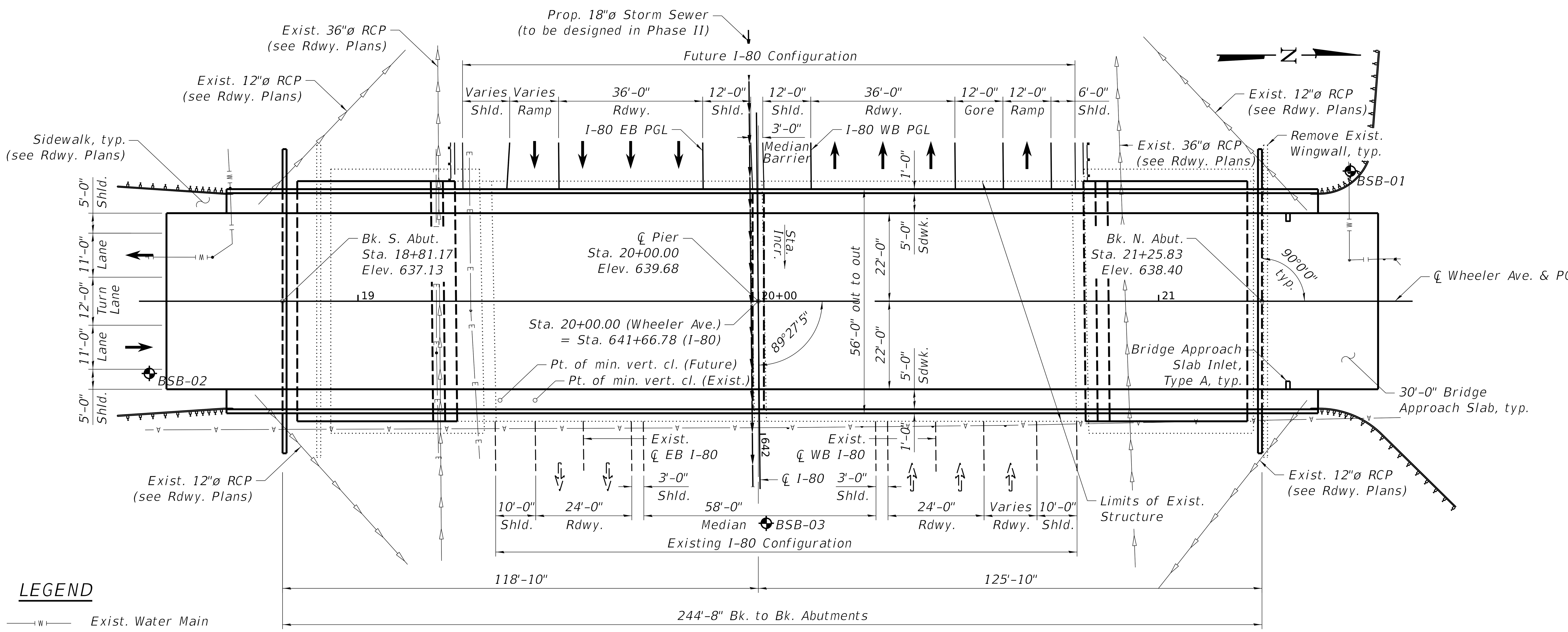
ELEVATION

LOADING HL-93

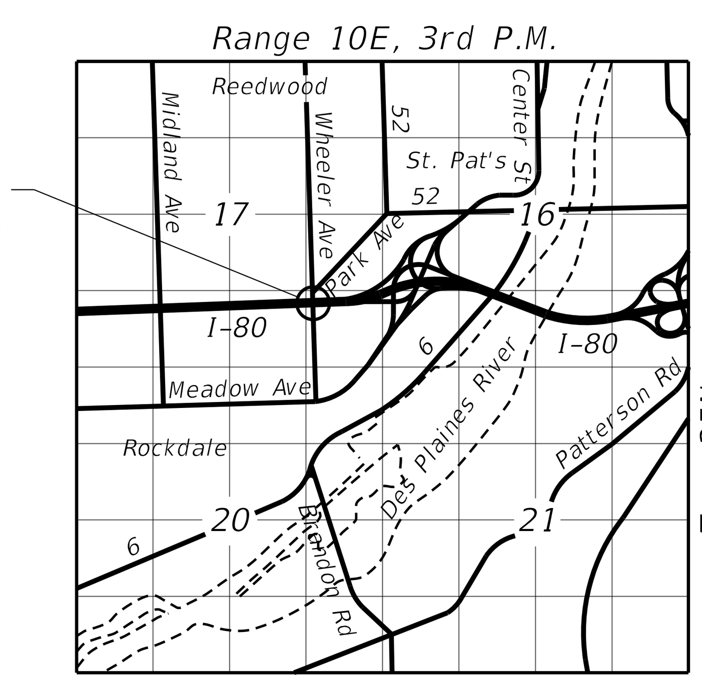
Allow 50#/sq. ft. for future wearing surface.

SEISMIC DATA

Seismic Performance Zone (SPZ) = 1
Design Spectral Acceleration at 1.0 sec. (SD1) = 0.068g
Design Spectral Acceleration at 0.2 sec. (SDS) = 0.126g
Soil Site Class = C



PLAN



LOCATION SKETCH

LEGEND

- Exist. Water Main
- Exist. Aerial Electric
- Exist. Street Lighting
- Soil Boring Location

GENERAL PLAN AND ELEVATION
WHEELER AVENUE OVER I-80
F.A.I. ROUTE 80 - SEC. XX-XXX
WILL COUNTY
STA. 20+00.00
STRUCTURE NO. 099-8324

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AECOM
303 EAST WACKER DRIVE, SUITE 1400
CHICAGO, IL 60601-5276
PHONE: (312) 373-7700 FAX: (312) 373-6800

USER NAME =	DESIGNED - DD	REVISED -
	CHECKED - ATB	REVISED -
PLOT SCALE =	DRAWN - DD	REVISED -
PLOT DATE =	CHECKED - ATB	REVISED -

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SHEET 1 OF 2 SHEETS

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
80		WILL	2	1

CONTRACT NO.

ILLINOIS | FED. AID PROJECT

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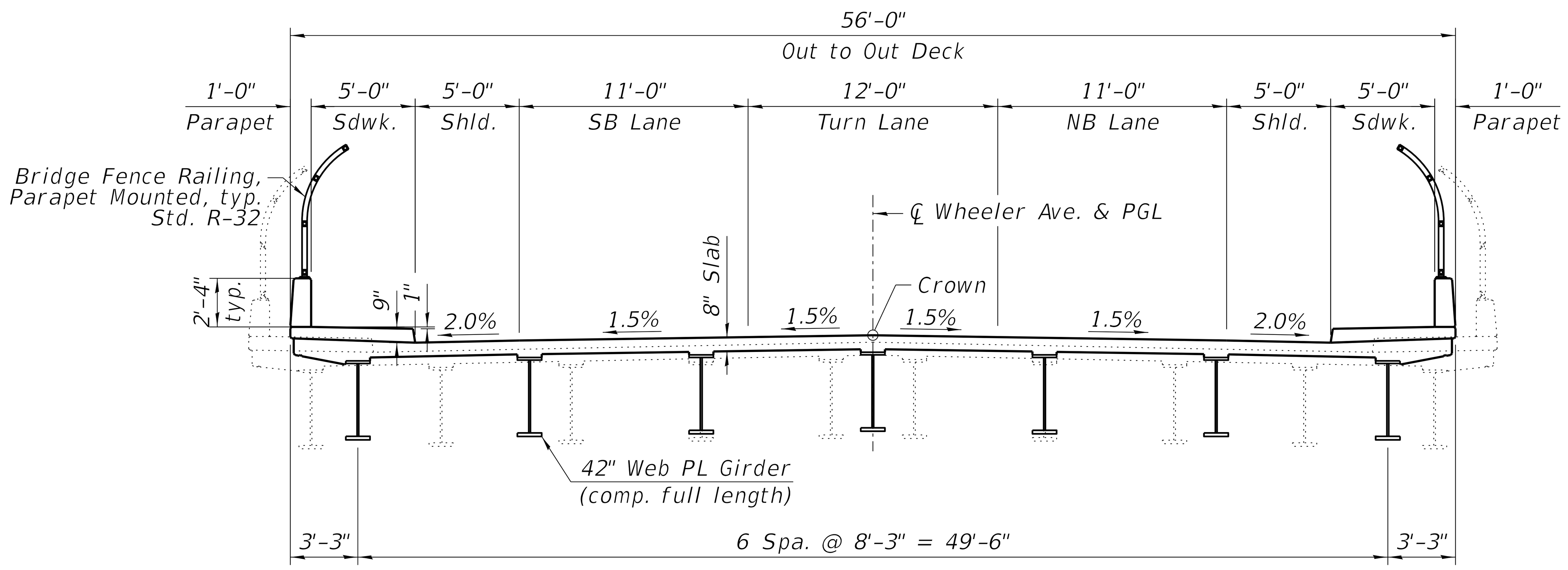
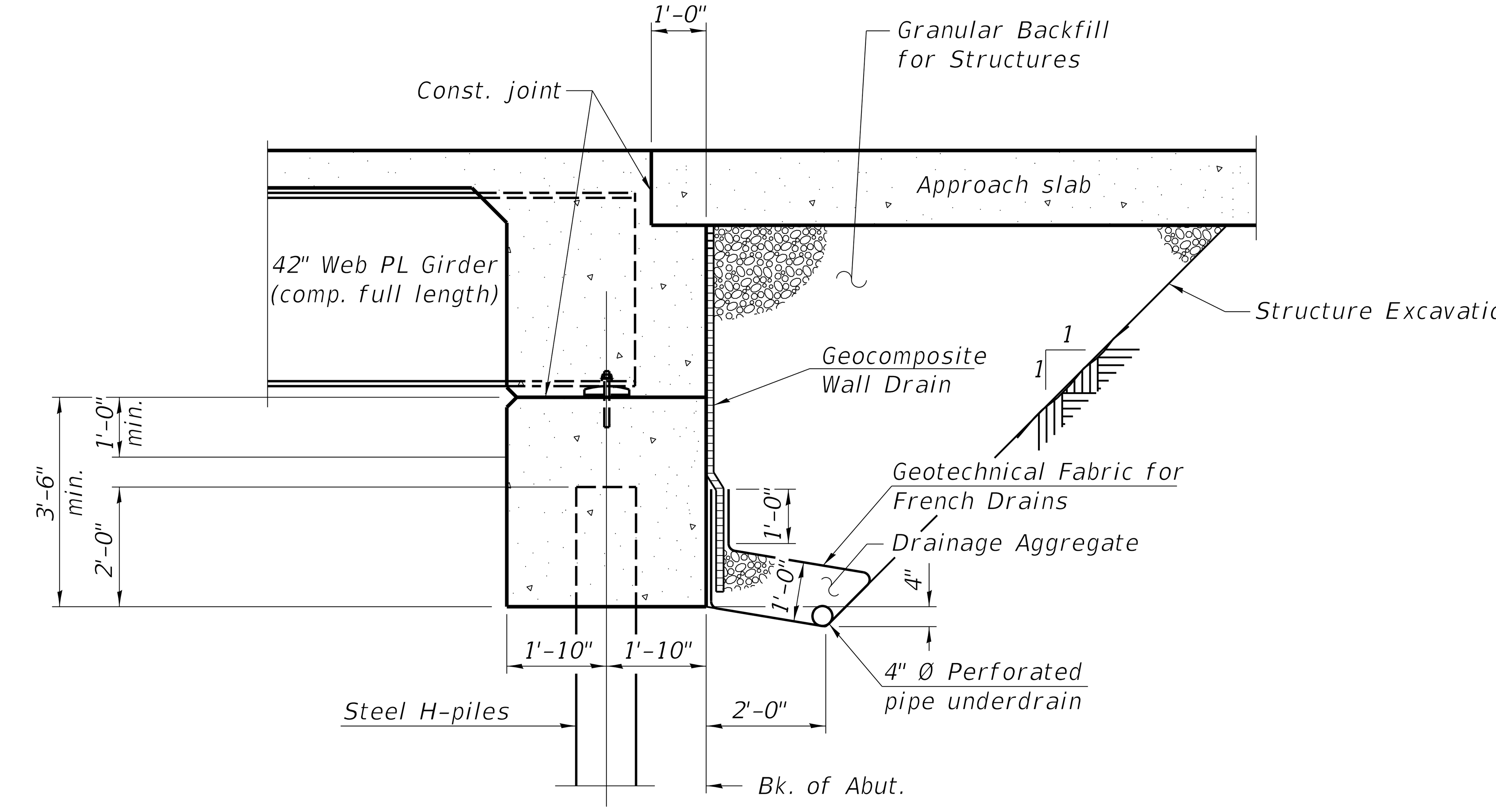


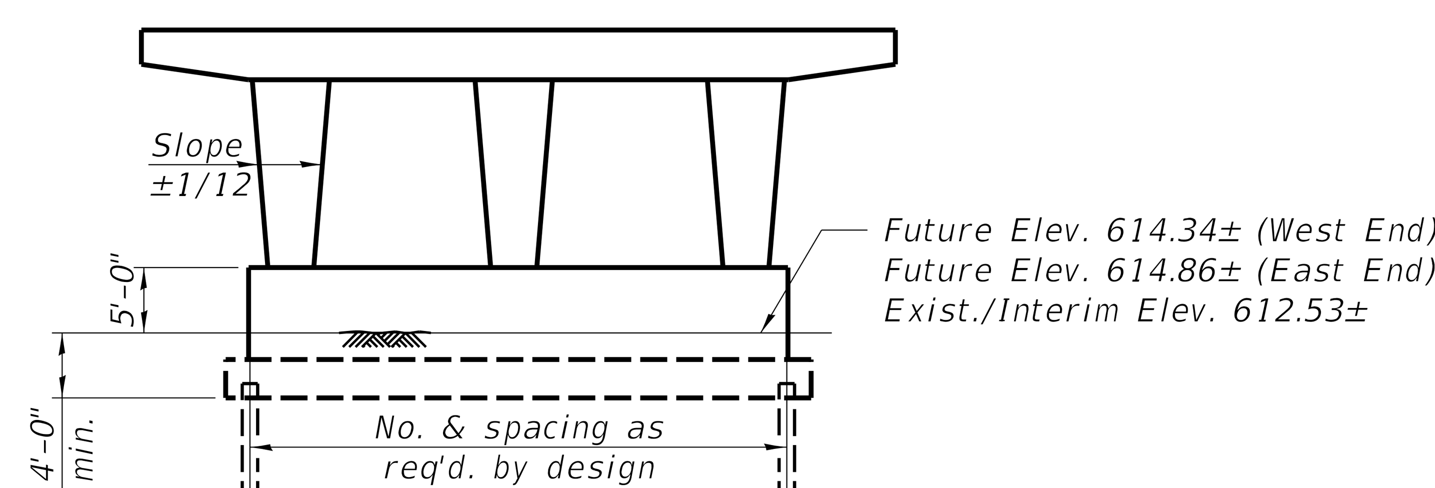
CROSS SECTION

(Looking North)

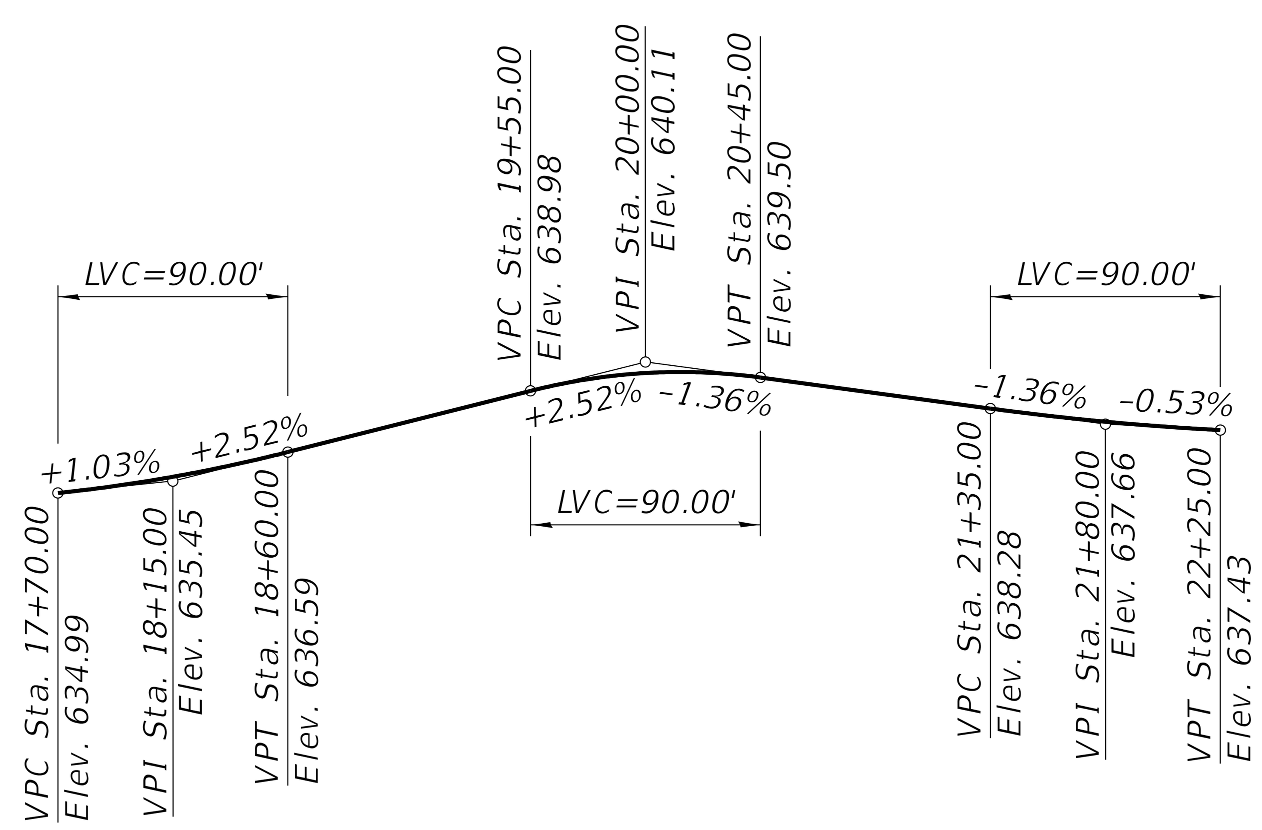


SECTION THRU INTEGRAL ABUTMENT

(Horiz. dim. @ Rt. ∠'s)

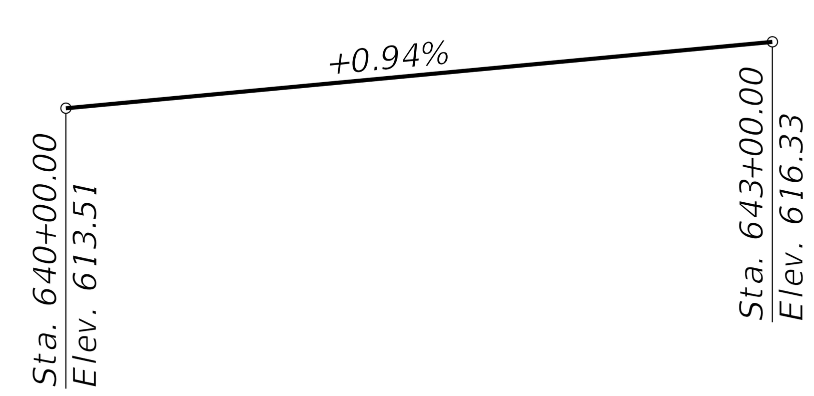


PIER SKETCH



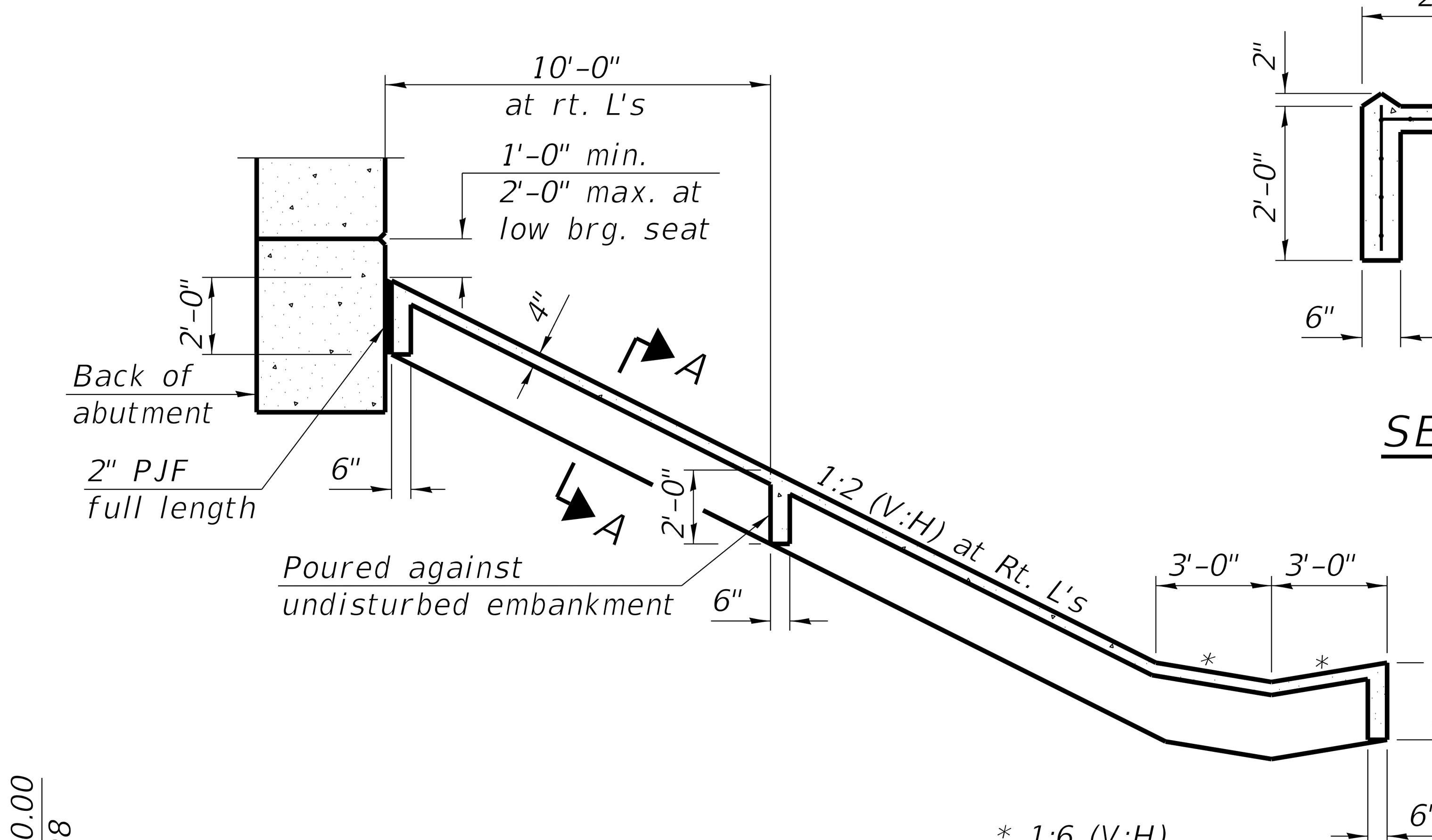
WHEELER AVE. PROFILE GRADE

(Along ℄ Roadway)

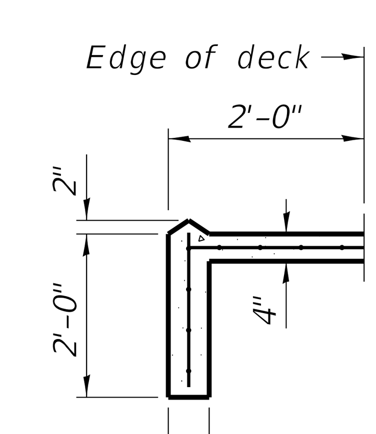


FUTURE I-80 PROFILE GRADE

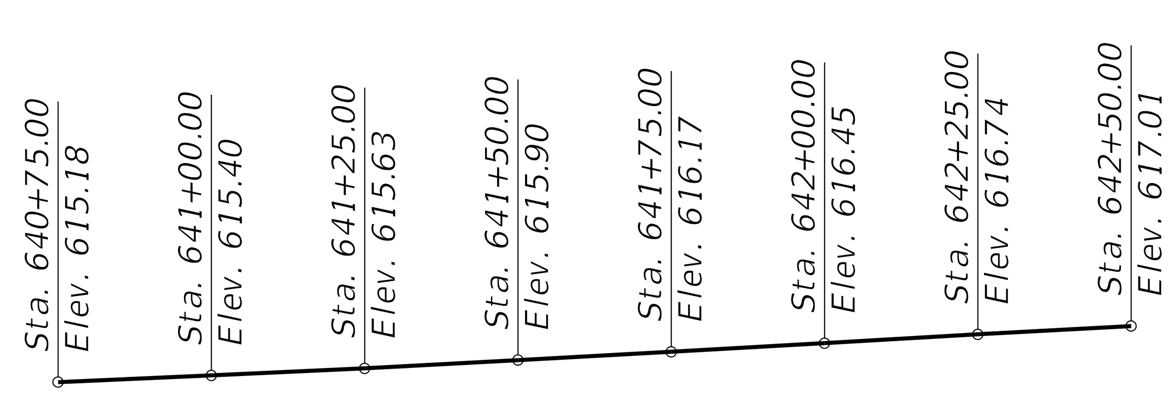
(Along I-80 EB & I-80 WB PGL's)



SECTION THRU CONCRETE SLOPEWALL

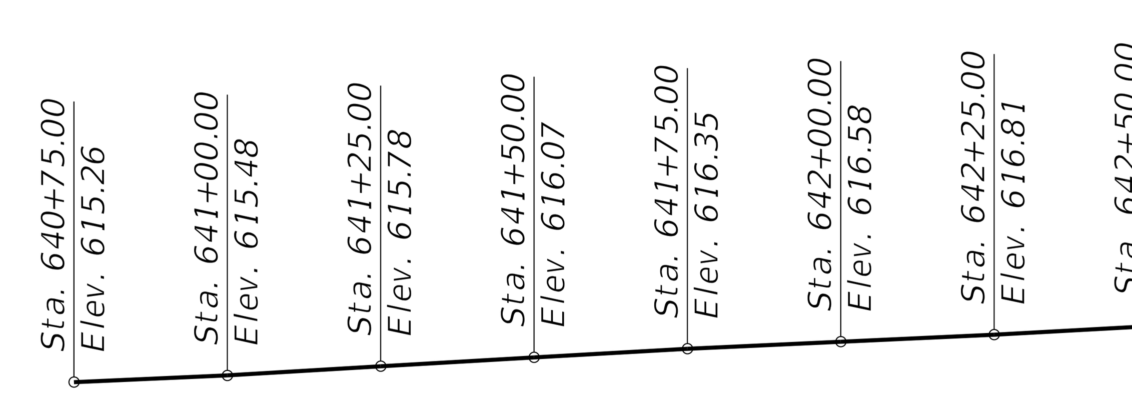


SECTION A-A



EXISTING EB I-80 PROFILE GRADE

(Along Exist. ℄ EB I-80)



EXISTING WB I-80 PROFILE GRADE

(Along Exist. ℄ WB I-80)

DETAILS

WHEELER AVENUE OVER I-80

F.A.I. ROUTE 80 - SEC. XX-XXX

WILL COUNTY

STA. 20+00.00

STRUCTURE NO. 099-8324

AECOM

303 EAST WACKER DRIVE, SUITE 1400
CHICAGO, IL 60601-5276
PHONE: (312) 373-7700 FAX: (312) 373-6800

USER NAME =	DESIGNED - DD	REVISED -
	CHECKED - ATB	REVISED -
PLOT SCALE =	DRAWN - DD	REVISED -
PLOT DATE =	CHECKED - ATB	REVISED -

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SHEET 2 OF 2 SHEETS

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
80		WILL	2	2
		CONTRACT NO.		
	ILLINOIS	FED. AID PROJECT		

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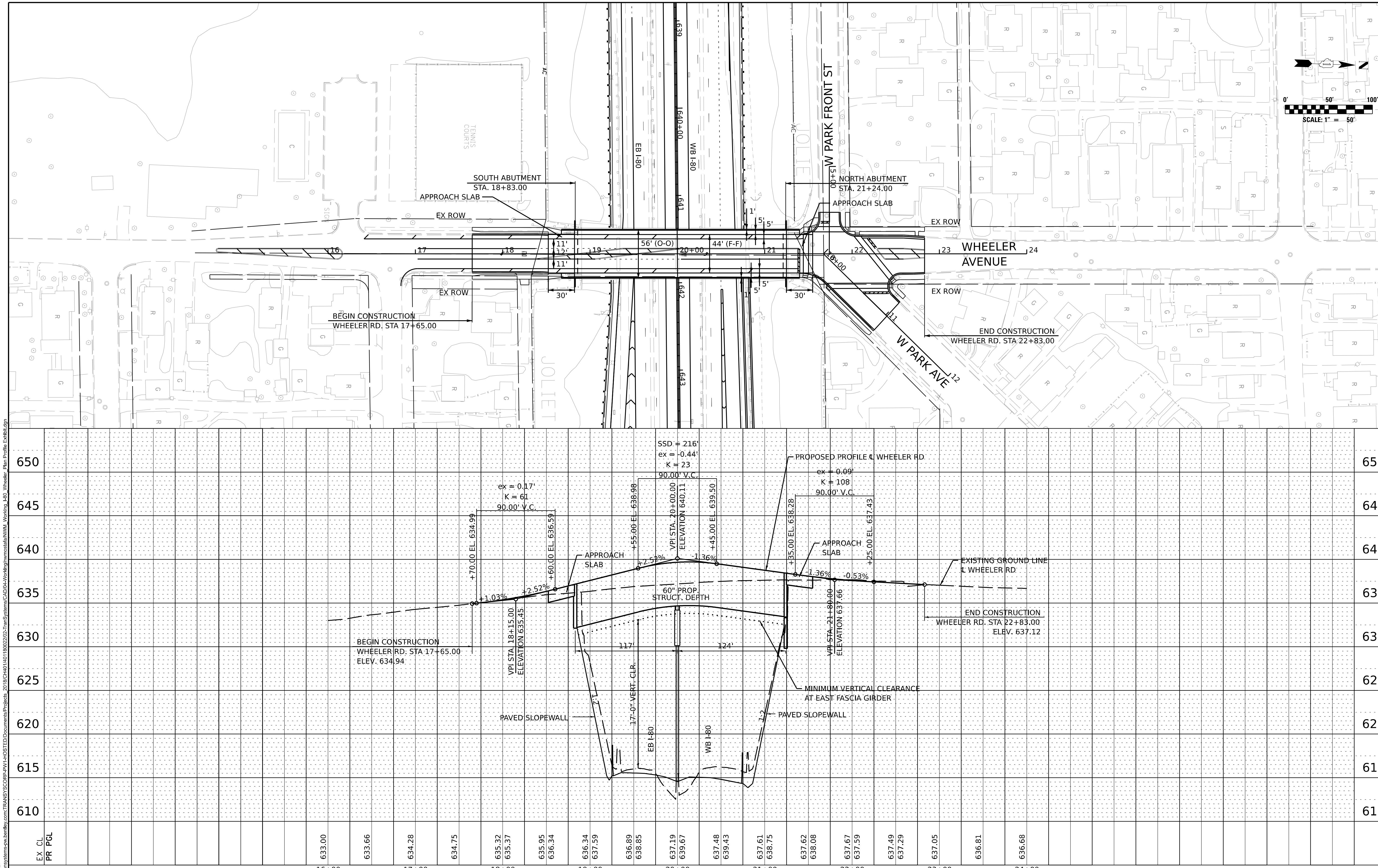

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STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

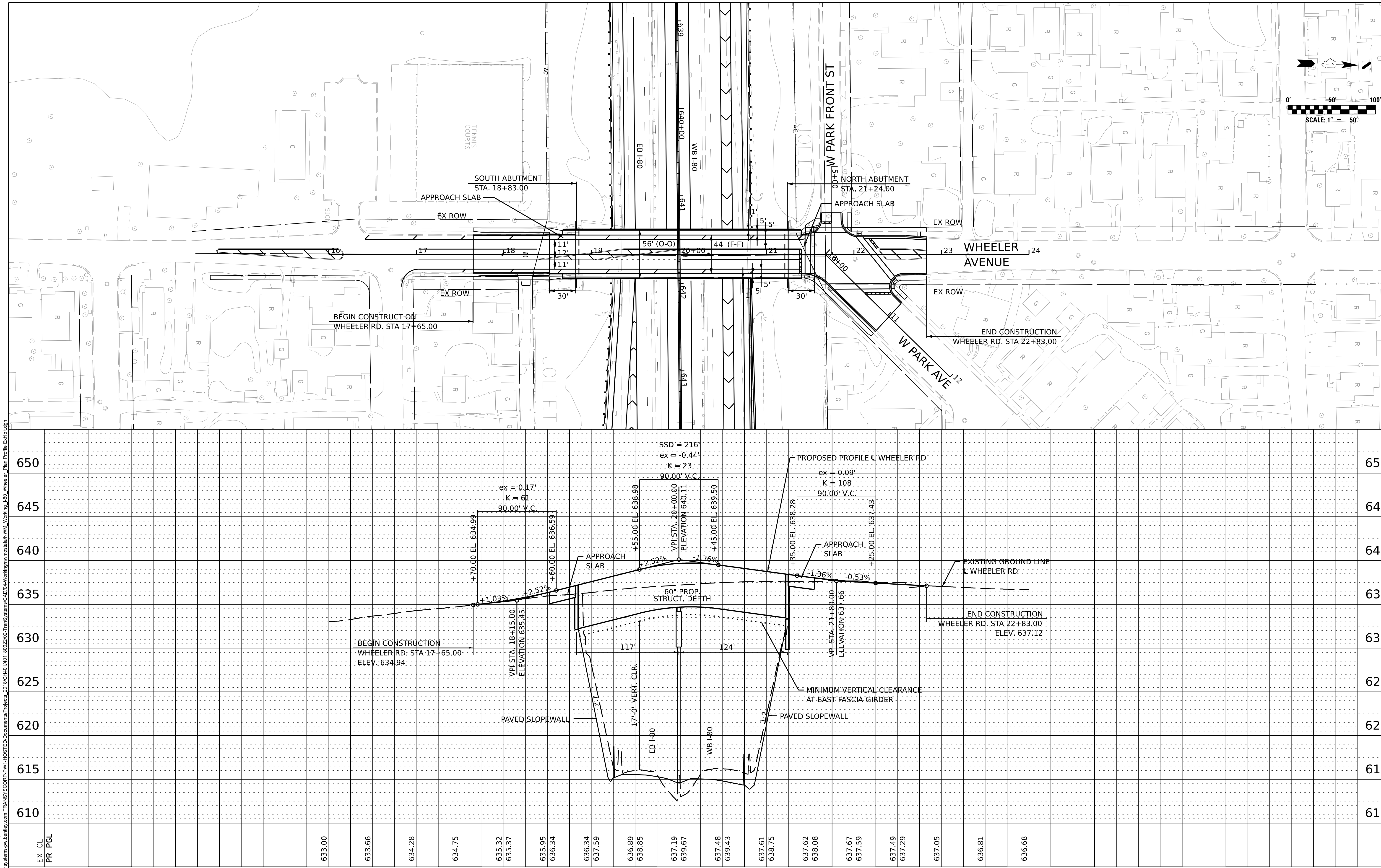
I–80 FROM RIDGE ROAD TO US ROUTE 30
WHEELER AVENUE PLAN AND PROFILE

SCALE: 1" = 50' | SHEET 1 OF 1 SHEETS | STA. 16+00 TO STA. 24+00

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
			1	1

CONTRACT NO.

ILLINOIS FED. AID PROJECT



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PLOT DATE = 8/10/2021	DATE -	REVISED -	

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

I-80 FROM RIDGE ROAD TO US ROUTE 30
CENTER ST RAMP D EXTENSION
WHEELER AVENUE PLAN AND PROFILE

SCALE: 1" = 50' | SHEET 1 OF 1 SHEETS | STA. 16+00 TO STA. 24+00

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
			1	1

CONTRACT NO.

ILLINOIS | FED. AID PROJECT

MODEL: Exhibit - Revised Geometry